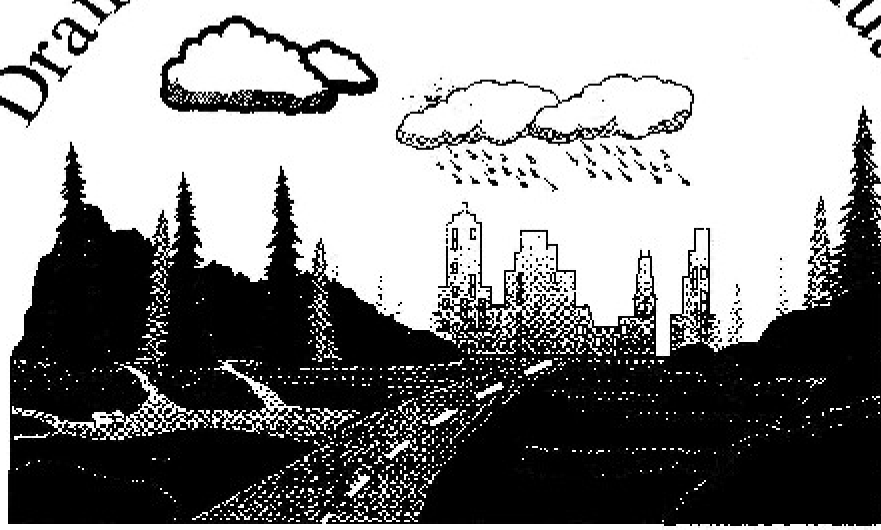
Ministry
of
Transportation
Ontario

MTO
Drainage Management Manual



Drainage and Hydrology Section
Transportation Engineering Branch
Quality and Standards Division

FOREWORD

The Ministry of Transportation of Ontario firmly believes in good drainage management practice in highway development projects. Good drainage management protects highway infrastructures, property owners and users against flood and drainage-related safety hazards. At the same time, good drainage management protects the land and water environments in watersheds impacted by highway projects and assists in conservation.

The ministry believes that good drainage management practice starts with practitioners and decision makers embracing an attitude of respect for the natural environment and willingness to work cooperatively with it in development of highway projects. Practitioners with an understanding of up-to-date concepts and principles of good drainage management practice as well as necessary professional skills to accomplish their work can achieve transportation objectives as well as providing for the natural environment. To these ends, the ministry has developed the *MTO Drainage Management Manual* to facilitate and direct drainage management practice within the ministry. It will be used by ministry staff of all levels as well as consultants working on provincial highway project assignments.

This manual should be used in conjunction with ministry directives which set objectives of practice and general design criteria. Also, this manual is intended to be used within the highway planning and design process and the class environmental assessment process.

Two existing publications, namely, *MTC Drainage Manual* of various dates from 1980 to 1988, and *MTO Drainage Management Technical Guidelines* dated November, 1989 are now replaced by this manual. Applicable materials in these two publications have been incorporated into the manual.

This manual does not set standards for parties and projects external to the ministry. However, the ministry hopes that it will be useful to land developers, municipalities, conservation authorities and their respective consultants in understanding the ministry's drainage management practice. It should also help external parties to understand concerns the ministry may have with regard to drainage management proposals affecting the provincial highway corridor.

This manual is the result of the efforts of many, including the MTO Drainage Management Manual Advisory Committee; the Editorial Panel and the writing teams. Participants included both ministry staff and external individuals and organizations. Input from regional offices has notably influenced the outlook and contents of the manual resulting in a document that reflects current operational issues and needs of users. The ministry wishes to thank all who were involved (names appear in the Acknowledgements inside the manual)

October 1997

DISCLAIMER

The *MTO Drainage Management Manual* (the manual) has been developed for use by the Ministry of Transportation of Ontario for its provincial highway projects. Other prospective users should determine for themselves whether the manual is applicable to their practices before they use the manual. The responsibility for the decision is the practitioners'. The ministry expressly disclaims responsibility for any inaccuracy or error which the manual may contain or for the fitness of the manual or the validity of the information contained in the manual for any particular purpose, or for any damage or loss which any person may suffer as a result of reliance upon any statement which the manual may contain.

Preface

The Scope of This Manual

This manual has been developed for use by the staff and consultants of the Ministry of Transportation of Ontario (MTO). It covers the practice of drainage management normally associated with the planning and design of highway projects. This manual deals with drainage practice issues such as:

- developing solutions to flood plain concerns associated with the selection of highway horizontal and vertical alignments;
- incorporating watershed drainage concerns when determining tradeoffs between highway alignments, property acquisition, and modifications of streams to accommodate highways; and
- using engineering knowledge of stream morphology to select suitable locations for bridges and culverts.

The manual provides methodology for the hydraulic design of a variety of drainage facilities. These include: roadside ditches, sewers, pavement and bridge deck drainage, stormwater ponds, bridges, culverts, stream channel works, and temporary erosion and sediment control works on construction sites.

Generally, applicable standards of practice are included in the discussion of the practice and design methodology. This includes guidance on issues such as acceptable design standards for hydraulic analysis of bridges and culverts. However, to maintain the flexibility of the document specific design policies and criteria are not included in the manual, since policies and criteria change more often than design methodology. Policies and criteria may also vary with geographical settings, and for a given project special conditions may require flexibility in setting the design criteria. Moreover, this manual may be used by parties external to MTO to whom MTO directives may not apply. MTO users should not be unduly inconvenienced by the absence of specific MTO design policies, referred to as "directives" in the manual, since all MTO offices maintain a complete, up-to-date set of policies which is readily accessible to all staff.

It should be noted that specific design objectives, criteria, and options for an individual highway project, including the drainage management components, will be established by the project through the class environmental assessment process. The material presented in this manual provides the general ground work for developing project specific requirements. It is intended to be read and used in this context.

Design tools such as computer models, and reference materials mentioned, but not included in the manual, are not part of this document. Such material should be acquired directly from the appropriate suppliers.

Transportation engineering is a multidisciplinary field of engineering of which drainage management practice is but one component. Therefore, users of this manual should use this document in conjunction with other applicable manuals and in consultation with the practitioners from the other disciplines. These disciplines include: highway geometric design, structural engineering, environmental planning, and landscaping, to name a few.

Finally, it is important to recognize that the *Drainage Management Manual* is formed around the three basic tasks (i.e. develop study options, preliminary design, and detail design) that are fundamental to the planning and design of highways and their drainage systems. The advantages of this "task-oriented" organization are as follows.

- Over time, the mode used to undertake and deliver the planning and design of highway projects may change. For instance, the planning and design of highways may be "out-sourced" to the private sector. Since the tasks associated with the planning and design of highway drainage systems do not change, all practitioners can use the manual, regardless of whether they are MTO staff or agents who act on behalf of MTO.
- Over time, the process that drives the planning and design of highways may change. Since the manual is not tailored to suit a specific step-by-step process, it can adapt to any process-oriented changes because the fundamental tasks will not change.

Drainage Management Practised in This Manual

Drainage management practised by this manual may be described by its *basic concept*, *objectives*, and *scope* of application.

The *basic concept* of drainage management in MTO is adapted from that suggested by the Ministries of the Environment and Energy (MOEE) and Natural Resources (MNR) in the two publications: *Integrating Water Management Objectives into Municipal Planning Documents* and *Subwatershed Planning*, 1993. The main point of this concept are as follows.

- Watersheds and subwatersheds are the basic Planning units for land use planning and resources management.
- The community of living things should be considered along with the physical and chemical factors which form the environment. The main premise is that a wholesome natural environment is achieved over the long term when the environmental considerations are balanced with social and economical relationships.
- The watershed planning unit includes all water processes and factors involved in the hydrological cycle.
- The subwatershed planning process emphasizes the following considerations: protection is preferred to mitigation; understanding of the interactions of the components of the natural environment is encouraged; and watershed planning is expected to be a multidisciplinary and consultative process.

The drainage management *objectives* of MTO include those required for the protection of the natural environment, as mentioned in the MOEE/MNR publications, and those required to fulfil the provincial transportation business mandate. The transportation objectives include:

- provide transportation infrastructures for the movement of goods and people;
- provide efficient and safe operating conditions of highways in wet weather;
- protect transportation infrastructure
- investments against damage by floods, scour, and other long-term hydrologic factors;
- protect watershed lands upstream and downstream of highway right-of-ways from drainage impacts attributable to highways; and
- achieve best return on investment for drainage management in transportation infrastructures.

In applying the watershed planning concept mentioned previously, drainage management in transportation engineering will focus its *scope* on watershed areas involving highways directly or indirectly. Watershed areas not involving highways are typically outside the scope of work. Similarly, the subjects of interest and concern are those arising directly or indirectly from highways. Broader subjects not related to a highway, such as creating a ravine picnic area, are typically outside the scope of work.

Suggestion for Effective Use of the Manual

This manual, by necessity, serves several groups of users and covers the types of drainage facilities usually encountered in transportation engineering. However, it is not necessary for the users to be familiar with the entire manual to get any specific design instructions. The editors have been mindful of the users' concerns and organized the manual in a way that hopefully will satisfy the largest number of users. The manual is divided into four parts, each part being a specific grouping of chapters. Within a chapter, the materials are further organized around each type of design task and are generally self-contained. The headings of the sections within a chapter are worded such that they facilitate locating the information quickly.

Part 1 of the manual deals with developing preliminary solutions and highlights the design considerations that should be taken into account early in the highway project stage. Technical details are kept to the minimum. This part can best be used by project managers and others involved in highway preliminary design.

Part 2 provides design methodology in a step-by-step approach to facilitate learning of the working details of the design methods. Many design examples are included to illustrate the application of these methods. This part is intended for users who need information on detailed analysis and design techniques.

Part 3 presents the engineering principles and theoretical background of the design methods discussed in Part 2. This part is a convenient reference providing explanations of the principles behind the design methods. It can be used on an "as needed" basis.

Part 4 contains design charts that are powerful and time-saving tools for use with the design methods presented in Part 2.

A *combined index* and a *glossary* are included in the manual to provide guidance for locating specific topics, and to define the major technical terms used in the manual. The glossary and the combined index are bound separately from the other four parts of the manual, so that the user may place them in the more frequently used part of the manual.

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The authors of the *Drainage Management Manual* acknowledge the contributions of past authors of the *MTO Drainage Management Technical Guidelines* and the *MTO Drainage Manual*, as significant portions of these documents were incorporated within the *Drainage Management Manual*. Specific references are noted in the appropriate chapter.

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Chapter 5: Bridges, Culverts and Stream Channels

Chapter 6: Temporary Sediment and Erosion Control

Ministry of Transportation

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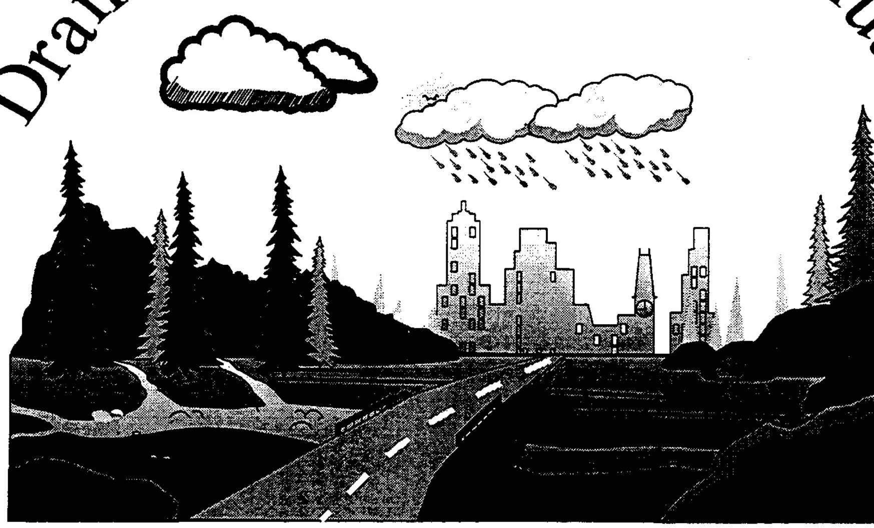
MTO Drainage Management Manual Part 1



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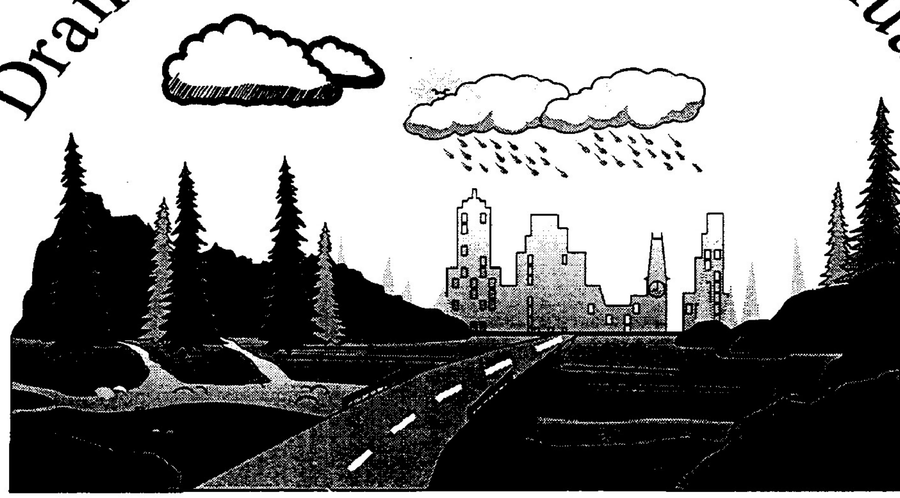


Chapter 1
Introduction to the Manual

Drainage and Hydrology Section
Transportation Engineering Branch
Quality and Standards Division

1 Table of Contents

Purpose of This Chapter

The purpose of this chapter is to introduce drainage management as it is associated with the highway design process for the Ministry of Transportation of Ontario (MTO). A discussion on the evolution of drainage management (i.e. watershed based approach) is presented to illustrate the changes that have taken place over time in Ontario. The discussion in this chapter is intended to illustrate the rationale for the publication of this new edition of the *Drainage Management Manual* and show the linkage between modern drainage management and highway planning and design.

The information in this chapter is also presented to:

- establish the purpose for requiring drainage management in highway planning and design; and
- provide a statement of the mainstream drainage management approach supported by MTO and adopted in this manual.

Modern Drainage Management

The Evolution of Drainage Management in Ontario

Past Practices (pre 1970's)

In the past, drainage management activities across Ontario focused on public safety and the protection of site specific capital investments (i.e. the prevention of flooding). This period of time was symbolized by the channelization of natural river/stream channels, and the construction of oversized drainage systems. Cumulative impacts that resulted from development were generally ignored. Consequently, natural receiving drainage systems (i.e. rivers, creeks, lakes) were affected by increases in flooding and erosion, as well as through degradation of water quality. Simply put, it was a period when the importance and function of natural receiving drainage systems was not well understood.

This utility-based design methodology was characterised by generally limiting impact assessments to the site of development. Drainage systems were designed to:

- convey rainfall runoff, as quickly as possible, off the surface and into storm sewers or ditches that discharge into the nearest receiving drainage system (i.e. river, stream, ditch, creek, etc.) ;
- minimize flooding of upstream properties; and,
- prevent flooding of the development.

Further Developments (late 1970's to early 1980's) - Stormwater Management

As adverse impacts of the common approach to drainage management became apparent, the province-wide drainage management practice was revised to include impact assessments that went beyond the site of development. The revised approach had an added focus towards the prevention of downstream flooding and erosion problems. Stormwater management techniques (i.e. dry detention facilities) were applied to reduce peak flow discharges from developments. This approach minimized the erosion and flooding potential of downstream receiving waters. Even though it was apparent that water quality problems still existed, stormwater was not perceived to be a contributor to water quality degradation. Correspondingly, water quality issues were not considered during drainage system planning and design activities.

Drainage management during this period was characterized by limiting discharges from the site of development to pre-development levels. The concept of “no increase” in peak flows was introduced (i.e. pre to post control). Although the overall drainage management practices improved, this approach continued to address drainage management issues on a site-by-site basis. As a result, cumulative impacts to the watershed were ineffectively assessed and other impacts to the watershed system (i.e. to aquatic or terrestrial habitat) were ignored.

Modern Drainage Management - Watershed-Based Approach (Since early 1990's)

In the early 1990's, water resource management agencies recognized that drainage management should be practised to account for all impacts within the watershed. In Ontario, the watershed planning approach emerged, with the key principles being that:

- the identification of overall watershed objectives should consider all physical, chemical, and biological parameters that are important to aquatic life and to human health ;
- the maintenance of "natural" hydrologic cycles is important to minimize alterations in habitat diversity, potential impacts to erosion and sedimentation processes, flooding levels, and groundwater supplies; and
- the maintenance of the "natural" river system is critical to the maintenance of "healthy" aquatic environments (i.e. concrete channel systems are not "healthy").

The watershed approach promotes implementation through a multi-disciplinary team (transportation engineers, biologists, landscape architects, water resources engineers and others). Provincially, the Ministry of Natural Resources and the Ministry of the Environment and Energy have shown long term benefits of the watershed-based approach and have promoted the change across Ontario.

Modern Drainage Management in the MTO

The evolution of drainage management within MTO has followed the same path as mainstream practice. In doing so, MTO, within its mandate as steward, owner, and regulator of provincial highways, has endeavoured to implement drainage practices that are beneficial to both the natural environment and the public.

This manual has been developed to reflect new developments in drainage management in Ontario, as well as in other jurisdictions, national and international. The *basic concept* of drainage management in the MTO is adapted from the approach proposed by the Ministries of the Environment and Energy (MOEE) and Natural Resources (MNR), in 1993, in the two publications: *Integrating Water Management Objectives into Municipal Planning Documents*, and *Subwatershed Planning*. The main points of this concept are as follows.

- Watersheds and subwatersheds are the basic planning units for land use planning and resources management.

- The community of living things should be considered along with the physical and chemical factors which form the environment. The main premise is that a wholesome natural environment is achieved over the long term when the environmental considerations are balanced with social and economical relationships.
- The watershed planning unit includes all water processes and factors involved in the hydrological cycle.
- The subwatershed planning process emphasizes protection over mitigation, an understanding of the natural environment, and a multidisciplinary and consultative approach.

In applying these basic concepts, drainage management in transportation engineering will focus its *scope* on watershed areas involving highways directly or indirectly. Watershed areas not involving highways are typically outside the mandate of MTO. Similarly, the subjects of interest and concern are those arising directly or indirectly from highways.

The main advantages of the watershed-based approach to drainage management, are as follows.

- Planning and design of highways with due regard to natural watershed characteristics, could avoid cumulative and long-term impacts on the watershed (i.e. receiving drainage system). Accordingly, this has the benefit of reducing the potential for over-control of stormwater and possible cost reductions in construction and maintenance through the integration or elimination of facilities.
- Where impacts are unavoidable, suitable methods of mitigation could be applied. Since it is the highway layout which will determine the overall effects on the watershed system, a thoroughly assessed highway plan may be as important, if not more important, than specific drainage management techniques.
- Due to the multi-disciplinary nature of drainage management, and the involvement of numerous groups to varying degrees, the watershed approach provides opportunities for the integration of drainage management issues of concern to regulatory agencies, early in the planning and design process. This avoids the complications associated with resolution of regulatory concerns at late phases of design, or on an *ad hoc* basis. Generally, complications can result in delays in receiving approvals which results in delays to the overall project schedule, and, ultimately, increased costs.

The MTO Drainage Management Manual

This manual strives to implement the modern drainage management approach while ensuring that appropriate guidance is provided to the highway drainage design practitioner. Specific objectives of the *MTO Drainage Management Manual* are presented below.

1. Strengthen the highway planning and design process by implementing the modern drainage management approach to:

- identify and screen drainage management issues at the initial stages of the planning and design process;
- allow flexibility to site highway infrastructures in appropriate locations (e.g. non-sensitive areas);
- plan drainage infrastructure considering existing topography (i.e. natural drainage patterns); and
- consider the use of alternative drainage management techniques while still maintaining the integrity of the highway infrastructure.

2. Recognize that drainage management is dynamic and has evolved into an integrated resource management approach.
3. Promote a consolidated team approach utilizing the numerous groups and disciplines that are involved in drainage management within MTO.
4. Ensure consistency in the application of drainage management, as it is practised across the province.
5. Ensure that regulatory concerns with highway drainage works are not addressed in an *ad hoc* manner.
6. Minimize potential liabilities associated with highway drainage works.

Drainage Management and the Highway Planning and Design Process

The process associated with the design of highway drainage management facilities is part of the highway design process, and decisions made regarding the drainage design are not made in isolation. The highway planning and design process is defined in the *Regional Planning and Design Project Management,* (MTO, 1992), and is presented in Figure 1.1. In recognition that changes in the specific details of the process may occur over time, Figure 1.1 has been reproduced in a more generic format. This format is presented in Figure 1.2. The linkage between drainage management and the Highway Planning and Design process is also presented in Figure 1.2.

The process for development of highway design, including drainage, can be divided into five main stages as follows:

- project initiation;
- identification of project objectives and criteria;
- study options;
- preliminary design; and
- detail design.

In developing the drainage design the following discussion describes the function of these stages.

Project Initiation

The first step in the planning and design process is characterizing the highway project. Characterization is generated outside the drainage management planning and design process. Characterization describes the physical characteristics of the constructed highway, highway operation, and highway maintenance. The extent and scale of both the highway project and the associated drainage management facility is usually compiled within the characterization. At this stage, drainage issues associated with the highway project, if any, are identified. A physical description allows a framework for identifying environmental impacts (natural, social, cultural and economic) to be considered when evaluating drainage management alternatives. Characterization of the highway project helps to determine criteria that are used in the evaluation of drainage management alternatives. Impacts can be determined for each project by comparing the physical and operating characteristics with the list of impacts described in Chapter 2.

Establishment of a data program very early in the planning and design process may prove to be an

Figure 1.1: The Highway Preliminary Design Process

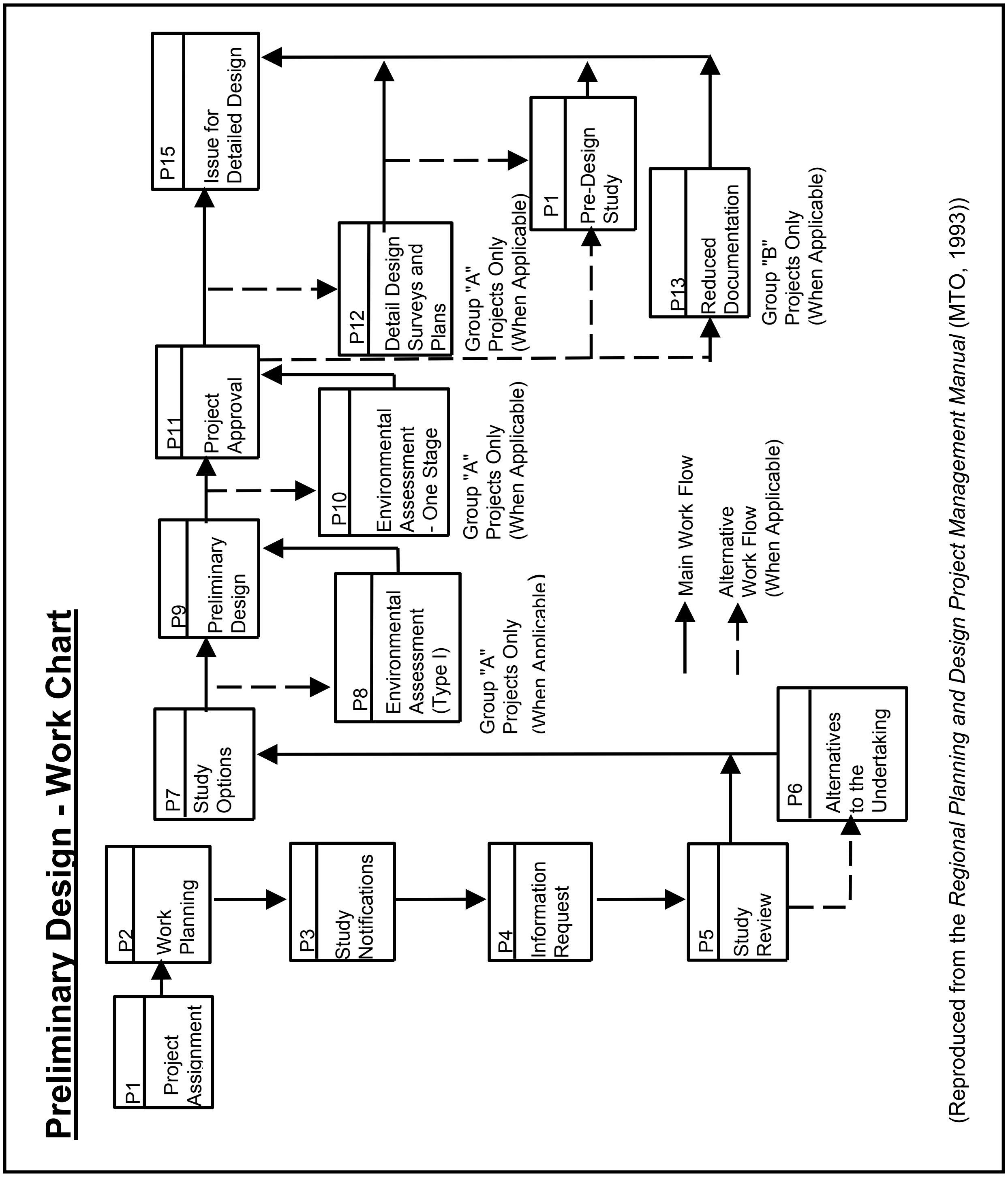
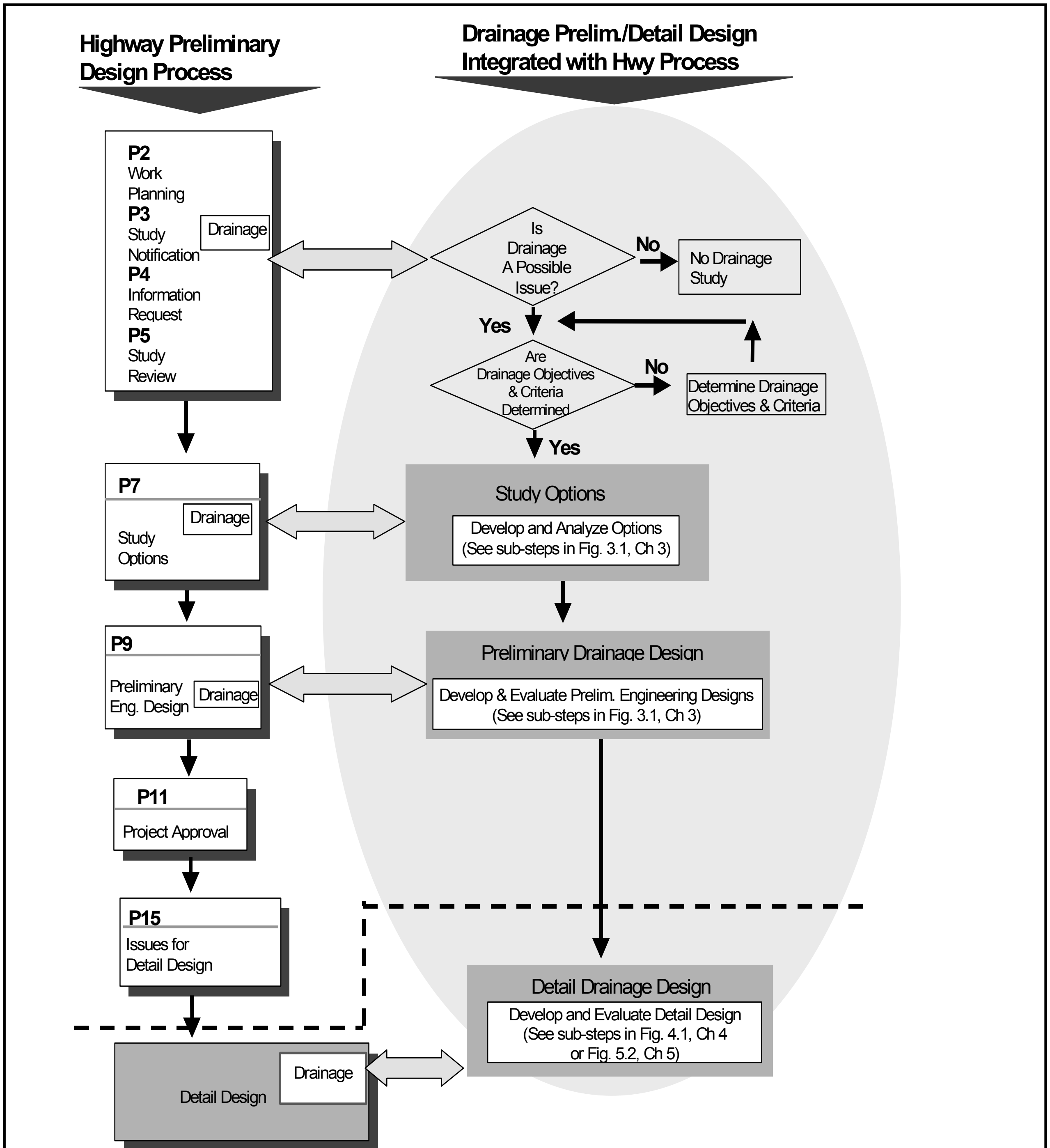


Figure 1.2: Integration of Drainage Management with the Highway Design Process



Notes: *MTO Regional Planning and Design Project Management* (1993).

efficient and comprehensive means of collecting different types of data, including drainage data.

This data will be required in defining the project characteristics. The data program will allow efficient retrieval, storage and manipulation of study data that becomes more specific as the planning and design process evolves.

Output from this stage includes physical characteristics of the project that will be utilized to identify potential highway and drainage management facility impacts.

Identification of Project Objectives and Criteria

An effective planning and design process requires a clear definition of the facilities objectives and criteria. Significant environmental impacts or high costs may result when inappropriate or vague objectives and criteria are established or informally defined.

Objectives and criteria must be defined together. Objectives are steps in achieving the goals and criteria are specific parameters applied to the design.

Objectives and criteria are based on the potential watershed impacts of a drainage management facility, laws, codes, policies, standards and guidelines. Criteria are developed through public and agency consultation. Some criteria exclude alternatives while others are used to evaluate the selected alternatives. At the end of this stage drainage objectives and criteria to guide the design process should be documented.

It should be noted that specific design objectives, criteria, and options for an individual highway project, including the drainage management components, will be established by the project through the class environmental assessment process. The material presented in this manual provides the general ground work for developing project specific requirements. It is intended to be read and used in this context.

Study Options

The first step in the design process is the development of study options for the different highway alternatives being considered. In each case a number of options may be feasible. Each of these options may have a number of associated impacts. Therefore, each option is analyzed and evaluated to eliminate those that do not satisfy the project objectives and criteria identified in the previous step. During this stage, additional information may be identified to assist in further analysis at later stages of development.

At the end of this stage, options that merit further investigation through preliminary design should be identified, additional information collected, and all the findings documented.

Preliminary Design

The preliminary design stage is a more detailed investigation of the drainage options identified at the study options stage. At this stage, however, a more detailed level of analysis and evaluation is needed to determine the most suitable option(s) that satisfy the design objectives and criteria prior to proceeding to detail design.

At the end of this stage documentation of the preferred design(s) may be prepared in the form of a preliminary design report.

Detail Design

At this stage the level of design analysis and evaluation of the preliminary design(s) is performed to select the preferred option. The level of analysis and evaluation is much more detailed, and the preferred option selected should satisfy the project objectives and criteria.

At the end of this stage the detail design of the preferred drainage management system is documented.

General Remarks

Points to consider when applying the process presented in Figure 1.2

- **The process may be iterative.** Design requirements, public concerns, scientific information, natural environment issues, and awareness of environmental processes may change. Consequently, depending on the project, objectives and criteria may have to be modified at any stage of the planning and design process, and new design options considered. This results in the design procedure being an iterative process.
- **The process is flexible.** All steps in the highway planning and design process are not always required nor necessarily followed in the specific order presented (i.e. as in Figure 1.2). The process is not rigid and need not be divided into separate stages. For instance, the completion of the study options and preliminary design may be done in one step if, for example, only one design option is being considered. The exact sequence of the process will depend on the scale and nature of each project.
- **The process includes drainage.** Drainage design is a part of the highway planning and design process. Correspondingly, the primary purpose of the *Drainage Management Manual* is to provide highway design practitioners with guidance on the design of drainage works in support of the highway planning and design process.
- **For rehabilitation, drainage works may be "the project".** In most instances, the highway project will involve the design and construction of highway elements associated

with widenings, realignments, and interchanges. However, in some cases the project may only involve drainage works. For example, a project may involve the analysis of culvert crossings to determine effectiveness, potential liabilities and long term maintenance requirements. Considerations, in such cases, will mostly be drainage related.

- **Long term monitoring** can determine the effectiveness of a facility in achieving the prescribed objectives and criteria. Monitoring should be conducted from the time of completion of a facility until abandonment. Monitoring includes reconnaissance (e.g. cursory visual observations) and detailed inspections (e.g. condition surveys, performance assessment, etc.). Monitoring can assess the difference between two inspections or cursory visual inspections. Not all MTO highway drainage facilities require detailed inspections. In some cases, reconnaissance may be adequate for ditches, catchbasins, gutters, etc.
- **Environmental impacts can be reduced** in future designs by modifying design criteria and parameters, to be based on long term monitoring results. In addition, impacts could be reduced through modifying operation and maintenance procedures.
- **The *Drainage Management Manual* is formed around the basic tasks** (i.e. develop study options, preliminary design, and detail design) that are fundamental to the planning and design of highways and their drainage systems. The advantages of this "task-oriented" organization are as follows.

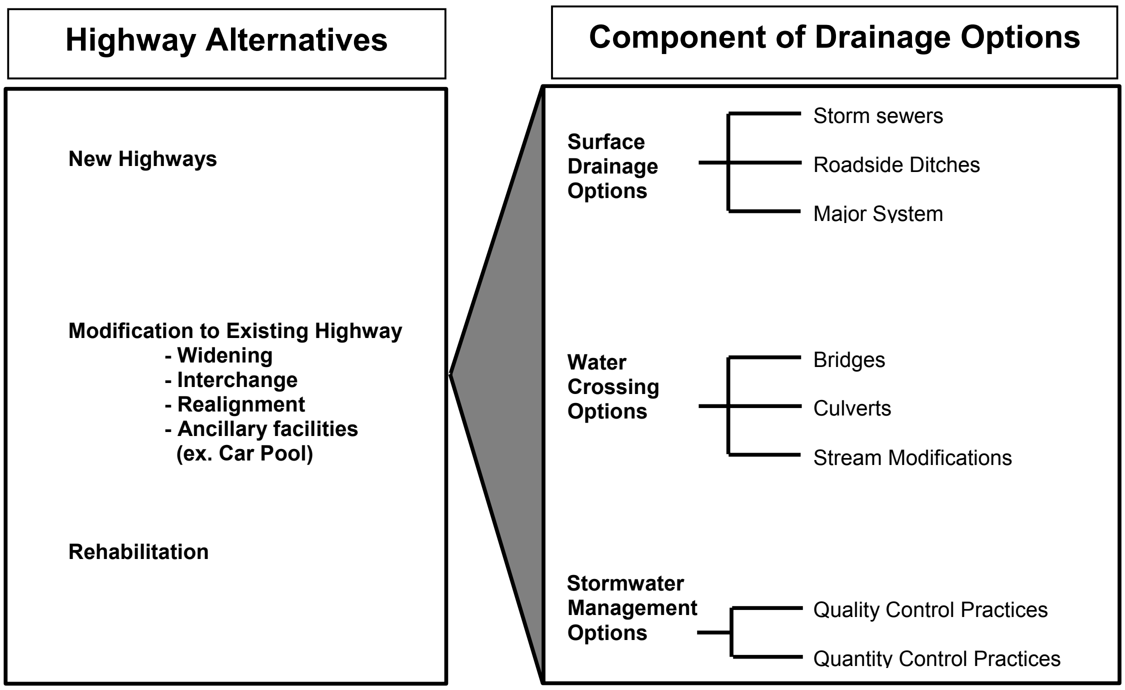
    - Over time, the mode used to undertake and deliver the planning and design of highway projects may change. For instance, the planning and design of highways may be "out-sourced" to the private sector. Since the tasks associated with the planning and design of highway drainage systems do not change, all practitioners can use the manual, regardless of whether they are MTO staff or agents who act on behalf of MTO.
    - Over time, the process that drives the planning and design of highways may change. Since the manual is not tailored to suit a specific step-by-step process, it can adapt to any process-oriented changes because the fundamental tasks will not change.

The Highway Project

The highway project, as illustrated in Figure 1.3, could include the following:

- horizontal and vertical realignments;
- widening;
- interchanges; and
- ancillary facilities.

Figure 1.3: Highway Alternatives and Associated Components of Drainage Options



Note 1: Highway alternatives are developed as outlined in the *Regional Planning and* Design Management Manual (MTO, 1993) which follows the Class Environmental Assessment procedure (see Figure 1.1).

The drainage options associated with these alternatives involve one or more of following components:

- water crossings (i.e. bridge, culvert, and stream modifications);
- surface drainage (i.e. storm sewers, roadside ditches, and major system); and
- stormwater management (i.e. quantity and quality control practices).

In a highway project considerations may be given to a number of alternatives which may include one, or combinations of the above (e.g. a new highway may include all of the above). Each highway alternative may have one or more associated drainage options.

Construction of drainage management works requires information that includes the following:

- size, shape, slopes and dimensions of the works;
- inlet/outlet configurations;
- location;
- property requirements;
- construction materials; and
- projected costs - capital, operating and maintenance.

Development of the above information will consider the following:

- environmental impacts of the provincial highway;
- location within the watershed;
- protection afforded by the drainage management works;
- environmental opportunities for improvement (i.e. corridor enhancement);
- impacts mitigated by the works; and
- impacts created by the works.

It is important to note that the environmental impacts associated with a highway project are not limited to the vicinity of the highway right-of-way, but can stretch far upstream and downstream. Therefore, the information requirements for drainage management and assessment of impacts may not be limited to the area in the vicinity of the right-of-way, but, in most cases, will include the entire watershed contributing to the particular area under consideration for drainage works.

Organization of This Manual

This manual provides information for the development of drainage designs for MTO. The manual is divided into four parts, and each part provides different types of information for different users. Table 1.1 shows the contents of each part and the intended users.

Part 1 of the manual assists Project Managers in making decisions pertaining to the development of drainage designs as part of a highway project. This part does not provide specific design details, but focuses more on the steps required for achieving the design of drainage works. Chapter 3 was included to outline the planning and design procedure for drainage works, and illustrate the types of activities and analysis associated with drainage designs.

Part 2 provides design details for the different components of drainage works associated with highway projects. It illustrates the design methodology with worked design examples. Part 4 includes the design charts referred to in the design examples and other parts of the manual.

Part 3 provides the theoretical background on which the design procedures in Parts 1 and 2 are based. This part is intended as a reference to provide further insight on the methods for analysis and design of drainage works discussed in the manual.

Table 1.1: Organization of the Manual and Intended Users

Part/Chapter	Chapter Title	Intended Users
Part 1 Chapter 1 Chapter 2 Chapter 3	**Developing the Drainage Design** • Introduction to the Manual • Developing Drainage Objectives and Criteria • Developing and Evaluating Design Alternatives	• Project Managers • Project Engineers • MTO Consultant
Part 2 Chapter 4 Chapter 5 Chapter 6 Chapter 7	**Design Methodology** • Surface Drainage Systems • Bridges, Culverts and Stream Channels • Temporary Sediment and Erosion Control • Data Sources and Field Investigations	• Design Engineers • MTO Consultants
Part 3 Chapter 8 Chapter 9 Chapter 10	**Reference Material** • Hydrology, Hydraulics and Stormwater Quality • Basic Stream Geomorphology for Highway Applications • Introduction to Soil Bioengineering	• Design Engineers • MTO Consultants • Other Users
Part 4	**Design Charts**	• Designers

Ministry
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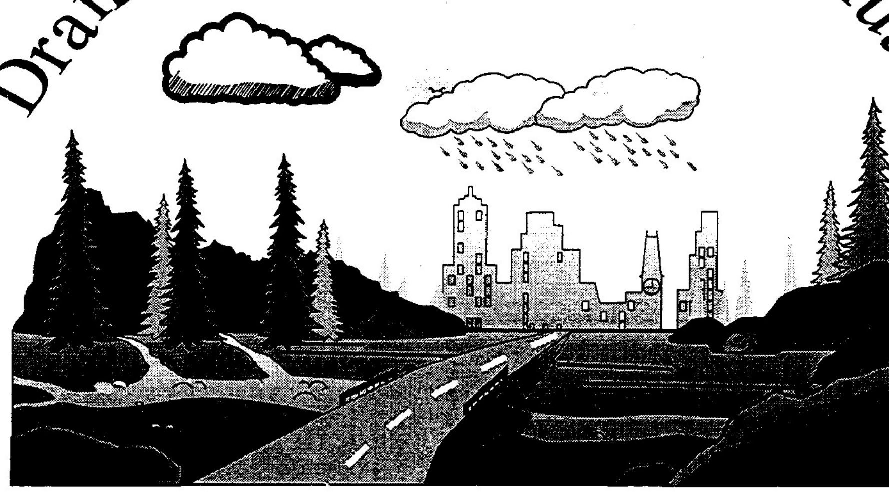
Ontario

MTO Drainage Management Manual



Chapter 2
Developing Drainage Objectives and Criteria

Drainage and Hydrology Section
Transportation Engineering Branch
Quality and Standards Division

2 Table of Contents

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Introduction

Purpose of This Chapter

The purpose of this chapter is to:

- present drainage considerations for developing highway drainage-related objectives and criteria ; and
- provide reference materials.

It should be noted that specific design objectives, criteria, and options for an individual highway project, including the drainage management components, will be established by the project through the class environmental assessment process. The material presented in this chapter is intended to be read and used in this context.

Definitions: Goals, Objectives and Criteria

Objectives are the premises needed to achieve the goals of a project. Criteria are developed from objectives and are used to measure the ability of an alternative to meet the objectives of a project. Many similar terms such as guidelines, policies, factors or constraints, may also be used to describe the actions outlined for objectives and criteria. Discussions with academic and practising engineers found that each of these terms means something slightly different to each individual. To minimize confusion in this chapter, only the terms objectives and criteria, as defined above, will be used.

Goals, broad targets that are to be achieved by the project, are generally linked to objectives and criteria. Although the highway planning and design process does not identify goals as being a specific part of the process, general goals can still be used to identify objectives for highway drainage works. Some general drainage-related goals are stated below and are only included for completeness.

- To convey upstream runoff through the highway corridor while minimizing upstream impacts, downstream impacts and impacts to the highway.
- To collect runoff from the highway corridor and convey it to the receiving drainage system while minimizing upstream impacts, downstream impacts and impacts to the highway.
- To meet drainage needs of the highway project.

Developing Objectives and Criteria

It is recognized that the highway project will define objectives and criteria. Therefore, the objectives and criteria associated with drainage management should be developed in conjunction with, and be incorporated into, those of the highway project. This approach is consistent with the integrated nature of the highway planning and design process, as it is presented in Figure 1.1 of Chapter 1. Some other general points regarding the development of drainage criteria or objectives are as follows.

- Drainage objectives should remain consistent throughout the highway planning and design process.
- Drainage criteria may change as the evaluation of drainage options progresses. At first, a small set of readily measurable criteria may be used. Once the short list of options is identified, more detailed information may be required to distinguish between the options. The overall procedure will be iterative.
- Drainage criteria will also vary according to the type of drainage option. For instance, drainage associated with the larger scale highway projects will require criteria to measure general impacts to watershed features, uses and characteristics. These criteria will emphasize avoidance of significant impacts.
- It is recommended that objectives and criteria be sorted according to impacts and then sorted according to the type of drainage works.

The planning and design of most drainage works will require an interdisciplinary team of professionals to establish and modify specific drainage objectives and criteria. Modification will generally result from consultation with the public and with regulatory agencies. Drainage objectives and criteria can be developed by considering the following:

- possible drainage impacts;
- common law principles;
- statute law requirements;
- documents supporting legislative mandates (e.g. policies, guidelines, manuals, etc.);
- consultation with the public and with regulatory agencies; and
- other needs (i.e. data collection, support studies and conflict resolution).

The confirmation of the selected drainage objectives and criteria should have the support of both the public and regulatory agencies. This reduces the likelihood of the drainage objectives and criteria being questioned after they have been applied, which can have both schedule and cost implications for a project.

Considering Possible Drainage Impacts

Background

Possible drainage impacts are presented as a preparatory step and can be used as a “screening tool” to provide the user with a quick method for identifying possible drainage impacts. The identified impacts can then be used as a guide for determining the scope and nature of the drainage objectives and criteria required for highway projects. Specific drainage objectives and criteria are determined by reviewing the considerations for developing drainage objectives and criteria that are presented in the subsequent sections. Appropriate guidance and information sources related to these considerations are presented within.

Due to the interdisciplinary nature associated with developing drainage objectives and criteria, it is intended that this chapter clearly outline the areas where consultation with, and involvement of, other professional disciplines is required. Information that is not directly related to drainage is included only for:

- information purposes;
- to familiarize the drainage practitioner with the language of other disciplines; and
- to familiarize the drainage practitioner with the issues that are shared between the different disciplines.

Solutions to impacts that are directly related to the drainage design are discussed in Chapter 3. For technical details refer to Chapter 4 or Chapter 5, in Part 2 of this manual.

Solutions to impacts, not directly related to the drainage design, are outside the scope of this manual and are discussed in other MTO documents such as:

- *the Environmental Manual, Fisheries (Working Draft)*; or
- *the Environmental Manual, Sediment and Erosion Control (Working Draft).*

When designing solutions to drainage related impacts, advantages gained through the interdisciplinary team approach cannot be overstressed.

Possible Drainage Impacts

Impacts, which can occur as a result of alterations in drainage associated with a highway project, are triggered by changes in one or more of the following on-site conditions:

- ground cover;
- topography;
- surface drainage systems; and
- contaminant inputs;

Table 2.1 provides examples of these possible changes in site conditions.

These changes have the potential to cause a variety of possible impacts. As an aid to understanding the potential impacts of highway drainage, impacts have been sorted into seven categories:

- hydrology;
- soil erosion;
- hydraulics and geomorphology;
- water quality;
- terrestrial biota;
- aquatic biota; and
- socioeconomic.

Table 2.2 presents a summary of possible drainage impacts along with possible causes. A more detailed listing of the causes and effects is presented in Appendix 2A, of this chapter. In reviewing Table 2.2 and Appendix 2A, it is important to note the following.

- There are a great many potential impacts associated with changes in highway drainage.
- The potential impacts are highly interdependent. A change in drainage can alter hydrology, which can alter river hydraulics and geomorphology, which in turn, can alter sediment loads and aquatic habitat. An understanding of the linkages between impacts is important when selecting the most appropriate mitigating measure.
- Some impacts are local; some are regional in nature.

Table 2.1: Examples of Changes in Significant Site Conditions

Ground Cover	The original vegetative cover(s) (trees, brush, agricultural crops, etc.) may be removed, and/or new ground cover(s) (pavement, rock, grasses etc.) may be installed.
Topography	Land slopes may be increased, decreased or altered in direction with excavation and/or filling to create highway subgrade, interchanges etc. and bridge abutments and approach ramps.
Surface Drainage System	The natural pattern of surface runoff and/or the continuity of overland flow paths may be altered by highway rights-of-way, profiles and barriers (safety guide rails, noise barriers etc.), and retention and deposition ponds. Physical characteristics of streams and waterways (length, cross-section size and shape, roughness etc.) may be altered in the vicinity of water crossings due to stream modifications/ diversions, temporary works (fording, coffer dams etc.) and design features such as abutments, piers, dykes and groynes.
Contaminant Inputs	The presence of highway traffic and road maintenance introduces the opportunity for many contaminants to enter the adjacent environment, including the drainage system (e.g. deicing compounds such as chlorides, sodium, calcium, ferric ferrocyanide, sodium ferrocyanide, and chromate of phosphate; nutrients and herbicides; grease, oil paraffins and heavy metals such as cadmium, chromium, copper, lead, magnesium, manganese, nickel and zinc from road runoff; minerals and chemicals from construction, refuelling areas, equipment storage areas, parking areas and stockpiles; chemicals and fuel from spills).

Table 2.2: Possible Drainage Impacts

Hydrologic Impacts

1 Increases in surface and other rapid runoff due to:
- installation of less permeable ground cover(s); and
- changes in the surface topography or the surface drainage system which expand the land area drained by surface flows.

2 Decrease in surface or other rapid runoff due to:
- changes in surface topography or the surface drainage system which reduces the land area drained by the surface flows.

3 Increase in the groundwater level and runoff , due to:
- the trapping or impoundment of surface runoff due to changes within the right-of-way such as road profiles, etc.; and
- reduction in evapotranspiration by removal of vegetation (with roots reaching the groundwater) from significant areas.

4 Decrease of groundwater level and runoff, due to:
- a reduction in opportunities for water to infiltrate the soil, by installation of impermeable ground cover rapid removal of surface runoff, etc.; and
- the drainage of shallow groundwater systems with the installation of surface and/or subsurface drains

Soil Erosion Impacts

1 Increased rates/ volumes of soil erosion, due to:
- decreased vegetation cover;
- increased ground slopes and/or slope lengths; and
- increased rates and/or volumes of surface runoff.

2.Increased amount of soil transported into waterways and streams, due to:
- increased soil erosion;
- increased surface runoff;
- improper construction techniques; and
- more extensive and efficient drainage systems.

Hydraulic and Geomorphologic Impacts

1 Higher flow velocities, due to:
- greater stream flows;
- narrowing of the stream cross-section;
- reduction in the channel roughness; and
- steepening of the channel gradient.

2 Lower streams velocities, due to:
- reduced stream flows;
- widening of the stream channel; and
- increase in channel roughness.

3 Deeper flow depths, due to:
- greater stream flows; and
- narrowing of the stream cross-section.

4 Shallower flow depths, due to:
- reduces stream flows;
- widening of the stream channel; and
- steepening of the channel gradient.

5 Increased sediment loads, due to:
- increased supply of sediment, from increased soil erosion, and/or the addition of bed material such as rock riprap etc.; and
- increased capacity to transport sediment, from increased stream flows, increased velocities, etc.

6 Decreased sediment loads, due to:
- decreased supply of sediment; and
- decreased capacity to transport sediment.

7 Degradation of the channel, due to:
- greater stream flow volumes and peaks;
- higher flow velocities; and
- deeper flow depths.

8 Aggradation of the channel, due to:
- lesser stream flow volumes and peaks;
- lower flow velocities; and
- shallow depths.

9 A change in the regime or form of the stream, due to:
- a change in the flow regime; and
- a change in the sediment regime.

Water Quality Impacts

1 Changes in the water chemistry of the streams and wetlands from the input of material(s), due to:
- loss of the material(s) from vehicles, such as oil, grease, trace metals, etc.;
- road maintenance procedures, such as road salting, pesticides spraying, etc.;

2 Changes in water quality due to alterations in physical processes only or in combination with the excess growth of plants, due to:
- changes in soil erosion rates;
- changes in the inputs of energy, e.g. increase in water temperature from the removal of riparian vegetation;
- increased nutrient supplies; and
- changes in organic matter quality and quantity, caused by the removal of riparian vegetation.

3 Changes in aquatic and surface sediment quality, due to:
- stormwater contaminated with trace metals and /or trace organics.

Table 2.2: Possible Drainage Impacts (continued)

Impacts on Terrestrial Biota

1 Losses or reductions in native or exotic plant species or communities associated with terrestrial ecosystems, due to:
- direct removal or injury of plant cover;
- changes in the micro-climate, such as the removal of tree canopy;
- changes in shallow groundwater systems; and
- increased soil salinity.

2 Expansion in the range of native or exotic plant species or communities associated with terrestrial ecosystems, due to:
- intentional or accidental introduction of exotic plant biota;
- changes in micro-climate;
- changes in shallow groundwater systems; and
- increased soil salinity.

3 Losses of animal species or communities associated with terrestrial ecosystems, due to:
- barriers in migration to animal movement, such as road profile changes within the right-of-way; and
- loss of habitat features required by animals.

4 Expansion in the ranges of animal species or communities associated with terrestrial ecosystems, due to:
- loss of predators; and
- increased habitat.

5 Disruption in the relationship between different components of terrestrial ecosystems, due to:
- planted roadside vegetation.

Impacts on Aquatic Biota

1 Losses or reductions in plant species or communities associated with tributary or wetland ecosystems, due to:
- direct removal or injury of plant cover.

2 Expansion in the range of native species or communities associated with tributary or wetland ecosystems, due to:
- increase supply of nutrients;
- loss of sensitive native species; and
- drainage works.

3 Losses or reduction in animal species or communities associated with tributary or wetland ecosystems, due to:
- acute stormwater toxicity;
- chronic or non-lethal effects associated with stormwater discharges or through contamination of aquatic sediments; and
- loss of habitat features required by animals.

4 Expansion in the range of animal species associated with tributary or wetland ecosystems, due to:
- degraded habitat.

5 Disruption in the relationship between different components of tributary or wetland ecosystems, due to:
- changes in the plant species and communities; and
- creation of artificial salt licks.

Socioeconomic Impacts

1 Loss of life and/or property, due to:
- flood caused by hydrologic and/or geomorphologic changes.

2 Loss or agricultural resources, due to:
- flooding;
- phytotoxicity of wet deposition, such as salt and particulate matter etc.; and
- alterations in drainage.

3 Loss of archeological and historic importance of native and non-native peoples, due to:
- loss or damage to sites of cultural or historic importance to native and non-native peoples.

4 Increased costs of water treatment, due to:
- impairment of water quality.

5 Loss of beneficial or recreational uses of terrestrial or aquatic biota, due to:
- loss of fish or bird species.

6 Loss of aesthetics, due to:
- loss of terrestrial and aquatic plants; and
- loss of walking or hiking paths.

7 Loss of biodiversity, due to:
- loss or reduction of animal and plant species.

Considering Common Law Principles

Common law is a body of principles based on long standing usages and customs, and on court decisions recognizing, affirming and enforcing such usages and customs. Common law, therefore, is largely a matter of precedent; the precedents can be modified as customs change and new practices arise. Common law principles:

- always apply unless enlarged, modified or superseded by statute law;
- give regard to current societal standards (i.e. what is deemed to be reasonable conduct in a given set of circumstances as well as reasonable expectations concerning what should or should not be foreseeable to a prudent person), and therefore are evolving; and
- are based on judgements rendered by the courts.

Since each particular highway project is unique and requires a slightly different solution, the development of drainage design criteria by lay persons interpreting previous court decisions may not always be appropriate. The practitioner is urged to obtain legal advice for all drainage matters that may lead to court judgements. Each drainage situation must be evaluated on its own merit. Sound judgement, proper design procedures and adequate documentation are very important.

When reviewing this section consider that:

- it is primarily written for MTO's staff (others may use this sections for reference, however, they are responsible for determining its applicability to their practice);
- it identifies the more important legal aspects of the design, construction and maintenance of highway drainage facilities, and provides a practical introduction to drainage law;
- it is not intended as an authoritative legal guide, but to give MTO staff a reasonable working knowledge of the subject;
- it should not be used to base legal advice or make legal decisions; and
- it is not intended as a substitute for legal counsel.

When reviewing common law principles, the type of water flow involved in any problem must be identified. Following this logic, common law, as it relates to highway drainage management, can be divided into the following subsections.

- Natural Watercourses:
  - Riparian Rights and Obligation;
  - Use of Water;
  - Interference with Natural Watercourses;
  - Diversions; and

  - Watercourse Crossings.

- Surface Flow:
  - Obstruction of Surface Flow;
  - Increase of Surface Flow;
  - Collection of Surface Flow; and
  - Surface Flow and the MTO.

- Subsurface Flow:
  - Underground Water in a Defined Channel; and
  - Underground Water not in a Defined Channel.

Table 2.3 presents examples of common law rights and obligations related to natural watercourses. Further details are discussed in Appendix 2B.

It is recognized that the obligations of a land owner who is seeking a sufficient outlet for drainage, have common law and statute law implications and could be included as part of the discussion on common law. However, for the purposes of this manual, the discussion on this issue has been limited to the presentation on statute law (refer to the subsequent section and to Appendix 2C).

Note: The discussion on Common Law contained within this chapter was taken from the original source (Madill, R.A., Harris, J.D., Tretjakoff, A. and McIlmoyle Q.C., A. B. (May 1980)) and modified.

Table 2.3: Examples of Common Law Rights/Obligations-Natural Watercourses

Attributes	Rights	Obligations

• Created by nature.
• Visible bed and confining banks.
• Sufficient flow to give it substantial existence.
• Riparian owners are those whose lands abut a natural watercourse. Bogs and swamps are not natural watercourses.
• Can include:
  • The valley through which a stream runs; and
  • A permanent artificial channel such as the Rideau Canal.
• Does Not Include:
  • Artificial ditches.
• Includes
  • Rainwater, melting snow, spring water;
  • Water diffuses over the surface and does not follow a defined channel;
  • Temporary and casual nature. Water disperses over the ground through percolation, evaporation or natural drainage; and
  • Does not gather or form any more definite body of water other than a bog or marsh.
	• Riparian owners have the following rights with regard to natural watercourses across their lands:
	  • To drain catchment area lands to the watercourse as long as flows do not exceed the capacity of the lower channel in its natural state;
	  • To discharge collected water via drains or ditches to the watercourse as long as flows do not exceed the capacity of the lower channel in its natural state;
	  • To make extraordinary use of water (e.g. operation of a mill) as long as the quantity and quality are not diminished;
	  • To have water flow to him/her in its natural state via the watercourse; and
	  • To use the stream water for domestic or natural purposes.
	• Higher lands can drain onto lower lands.
	• A lower land owner does not have to receive water from higher lands as long as the lower land owner does not injure an adjacent land owner when exercising the right not to receive surface water.
	• A land owner :
	  • Can collect and drain surface water but must have a sufficient outlet.
	  • Can dig a pond to retain water;
	  • Can build a dyke as long as the dyke does not collect or concentrate flow; and
	  • Can raise the level of the lower land above the higher land. Cannot direct high land water to lands that did not have water before.
		• May not bring waters that have not fallen within the watershed to the stream.
		• May not sell or assign the right to drain into a watercourse.
		• To ensure a sufficient outlet where there is interference with a natural watercourse.
		• To obtain a water taking permit from MOEE for withdrawals greater than 50,000 L/day.
		• Enclosures cannot reduce the watercourse capacity.
		• Must accept increased flows as long as it results from reasonable use.

Considering Statute Law Requirements

Statute law is established by a legislative body and set down in a formal document. Statute law can evolve (enlarged and modified) from common law (court made law) to correct inadequacies in common law. There are many statutes containing provisions which relate to drainage. Some statutes bind the Crown, while others do not. The statutes that relate to highway drainage, binding the Crown and applicable to MTO, are identified on Table 2.4. For a more detailed discussion on each statute, refer to Appendix 2C.

Since each particular highway project is unique and requires a slightly different solution, the development of drainage design criteria by lay persons interpreting statute law requirements may not always be appropriate. The practitioner is urged to obtain legal advice for all drainage matters that may lead to court judgements. Each drainage situation must be evaluated on its own merit. Sound judgement, proper design procedures and adequate documentation are very important.

When reviewing this section consider that it:

- identifies the more important legal aspects of the design, construction and maintenance of highway drainage facilities, and provides a practical introduction to drainage law;
- is not intended as an authoritative legal guide, but to give MTO staff a reasonable working knowledge of the subject; and
- should not be used to base legal advice or make legal decisions.

Administration of Statute Law

Federal and provincial statues provide for the administration of the requirements of legislation through government agencies in the form of federal departments, provincial ministries, conservation authorities, crown corporations, boards, and municipal or regional departments. Administrative authority is sometimes delegated from a higher to lower level of government. The agencies with which authority rests are the “mandated agencies”. In addition to the role of the mandated agency in granting approvals, other agencies may be assigned the role of a commenting agency in the approval review process.

Table 2.4: List of Statutes[1]

Title	Binding MTO[2]	Ministry Responsible
Canadian Environmental Assessment Act, R.S.C.	Yes	DOE
Fisheries Act, R.S.C., 1985, F-14	Yes	DFO
Navigable Waters Protection Act, R.S.C. - 1985, N-22	Yes	TC
Bridges Act, R.S.O. 1990, B.12	Yes	MTO
Environmental Assessment Act, R.S.O. 1990, E.18	Yes	MOEE
Environmental Protection Act, R.S.O. 1990, E.1	Yes	MOEE
Interpretation Act, R.S.O. 1990, I.119	Yes	ATG
Limitations Act, R.S.O. 1990, L.15	Yes	ATG
Public Transportation and Highway Improvement Act, R.S.O. 1990, P.50	Yes	MTO
Ontario Water Resources Act, R.S.O. 1990, O.40	Yes	MOEE
Beds of Navigable Waters Act, R.S.O. 1990, B.4	No	MNR
Conservation Authorities Act, R.S.O. 1990, C.27	No	MNR
Drainage Act, R.S.O. 1990, D.17	No	OMAFRA
Lakes and Rivers Improvement Act, R.S.O. 1990, L.3	No	MNR
Local Improvement Act, R.S.O. 1990, L.26	No	MMAH
Municipal Act, R.S.O. 1990, M.45	No	MMAH
Planning Act, R.S.O. 1990, P.13	No	MMAH
Public Lands Act, R.S.O. 1990, P.43	No	MNR
Tile Drainage Act, R.S.O. 1990, T.8	No	OMAFRA

Abbreviations:

DOE	Department of the Environment
DFO	Department of Fisheries and Oceans
TC	Transport Canada
MMAH	Ontario Ministry of Municipal Affairs and Housing
ATG	Ontario Ministry of the Attorney General
MNR	Ontario Ministry of Natural Resources
OMAFRA	Ontario Ministry of Agriculture Food and Rural Affairs
MOEE	Ontario Ministry of the Environment and Energy
MTO	Ontario Ministry of Transportation
RSO	Revised Statutes of Ontario
RSC	Revised Statutes of Canada

Notes: [1] Since legislation may change over time, the statutes listed in Table 2.4 may also change. To ensure accuracy, always refer to the official statute.

[2] One of the exceptions to the rule that a statute is generally presumed not to bind the Crown is that a statute will bind the Crown if the Crown seeks to take the benefit of the statute. In other words, one must take the burden with the benefit.

Agency Mandates

There are many statutes at the three levels of government which affect drainage management in the Province of Ontario, and the result is many agency mandates. A sample summary of agency mandates is provided in Table 2D.1 in Appendix 2D. In general, the mandates of the following agencies may implicate drainage management.

- Canada Department of Environment
- Canada Department of Fisheries
- Canada Department of Transport
- Ontario Ministry of Environment and Energy (MOEE)
    - MOEE Approvals Branch
    - MOEE Regional Offices
    - MOEE District Offices
- Ontario Ministry of Natural Resources (MNR)
    - MNR Regional Offices
    - MNR District Offices
- Ontario Ministry of Municipal Affairs and Housing (MMAH)
    - Development Standards
- Ontario Ministry of Intergovernmental Affairs (MIA)
- Ontario Ministry of Agriculture, Food and Rural Affairs (OMAFRA)
- Ontario Ministry of Culture and Communication (MCC)
    - Heritage Sites
- Ontario Ministry of Tourism and Recreation (MTR)
    - Park Lands
- Ontario Ministry of Health
    - Local Health Unit/Medical Officer of Health
    - On-site Sewage System
- Ontario Municipal Board
- Ontario Environmental Assessment Board
- Ontario Environmental Appeal Board
- Ontario Hydro
    - Hydro Lands
- Conservation Authorities
- Regional/Local Municipalities
    - Councils
    - Planning Boards
    - Parks Departments
    - Works Departments

Note: The discussion on Statute Law contained within this chapter was taken from the original source (Madill, R.A., Harris, J.D., Tretjakoff, A. and McIlmoyle Q.C., A. B. (May 1980)) and modified.

Considering Documents Supporting Legislative Mandates

The mandates of agencies, drawn through statutes, are implemented through the use of supporting documents. Approvals are generally sought in the context of these documents. Some forms of these support documents are discussed below.

- **Policies** such as MTO Directives and policies of other agencies may implicate drainage works. An example of a Provincial Policy document is the *Natural Heritage Policy Statement.*
- **Protocols** are agreements between two or more government ministries or agencies to define legal and administrative processes for MTO undertakings. An example of a protocol is the MTO/MNR/DFO Fisheries Protocol.
- **Guidelines and manuals** are prepared by various ministries to explain how a subject matter should be addressed, and provide resource reference information and suggested approaches.
- **Codes/Standards** must be followed when generating water crossing options. The *Ontario Highway Bridge Design Code* (OHBDC) is an example of a code.
- **Drainage plans** include watershed plans, subwatershed plans, master environmental servicing plans, and site plans that are prepared by agencies external to MTO. Criteria and objectives can be abstracted from these documents. Only plans that have been endorsed by MTO should be used to abstract objectives and criteria. Watershed or subwatershed plans, for areas that are applicable to MTO right-of-ways, typically include the following information that could be utilized to develop criteria:

    - areas that must be protected, rehabilitated and enhanced;
    - areas that can be developed in a manner compatible with subwatershed objectives;
    - policy/guidelines to direct development planning and design;
    - directives for stormwater management and groundwater plans, and other studies/designs for specific areas; and
    - design, function, siting and timing of facilities.

    In addition to criteria, watershed/subwatershed plans will undertake baseline inventories that may provide information on areas that could potentially be impacted by MTO highways and drainage systems. Baseline inventories include stream erosion, water quality, terrestrial habitats, groundwater, wetlands, environmentally significant and sensitive areas, woodlots, etc.

Documents Supporting MTO Legislative Mandate

MTO has adopted procedural directives, policies, protocols and manuals to ensure that the planning, design, construction and maintenance of highway facilities proceeds in a consistent, efficient, safe and responsible manner. The practitioner is encouraged to keep an updated list of directives that may provide information with regards drainage objectives and criteria.

Table 2.5: List of MTO Drainage Directives, Manuals and Protocols

Directive, Guidelines or Manual	Purpose
Directives:	
• PHY B-63	MTO Participation in Works under the Drainage Act
• PHY B-100	MTO Design Flood Criteria
• PHY B-217	Private Piped Drain on the Highway Right-of-Way
• PHY B-237	MTO Drainage Management Policy and Practice
Manuals:	
• Drainage Management Manual	Drainage management planning and design
• Planning and Design Project Management Manual	Highway planning and design process
• Environmental Manual, Fisheries (Working Draft)	Fish habitat measures
• Environmental Manual, Sediment and Erosion Control (Working Draft)	Process for practising sediment and erosion control during construction
Protocols:	
• MTO / MNR / DFO Fisheries Protocol	Protocol outlining Fisheries Act requirements

Documents Supporting Legislative Mandate of Regulatory Agencies

Documents that support the legislative mandates of the various regulatory agencies are numerous. Regardless, these documents may be useful when developing drainage objectives and criteria for the highway project. Table 2.6 presents the regulatory agencies that have useful support documents. Table 2E.1 in Appendix 2E focuses on the more common forms of the various support documentation applied by the mandated agencies at the various levels of government.

Table 2.6: Regulatory Agencies with Documents Supporting Legislative Mandates

Federal Level	Provincial Level	Municipal Level
Department of Fisheries and Oceans	Ministry of Environment and Energy Ministry of Natural Resources Ministry of Municipal Affairs and Housing Ministry of Agriculture Food and Rural Affairs Joint MOEE/MNR Conservation Authorities Joint MNR/MOEE/MMAH/MTO/ACAO[1]/UDI[2]	Local Municipalities Regional Municipalities

Note: [1] Association of Conservation Authorities of Ontario
[2] Urban Development Institute

Considering Consultation with the Public and with Regulatory Agencies

Consultation with the public and with regulatory agencies can be used to develop drainage objectives and criteria, and can identify conflicting external agency criteria. Regulatory agency/public consultation can provide the following:

- opportunities for agency/public concern and comment on drainage issues;
- additional project information related to drainage;
- property owner needs that have to be addressed by the drainage works; and
- identification of required drainage commitments to agencies and the public.

Consultation normally takes place within the highway planning and design process. The information developed during this process comes from a variety of sources. The establishment and modification of facility objectives and criteria will probably be developed through negotiations with external federal, provincial and municipal agencies, internal MTO offices, private individuals and corporations.

Considering Other Needs

Detailed Field Inventories and Data Collection

Detailed field inventories and data collection may be used to supplement information abstracted from information reviews, and to confirm discussions with external agencies and the public. For all projects, it is expected that there may be some form of field investigation. Investigations for the smaller projects will generally be limited to the cursory examinations of the site. The investigations may identify information that is needed and was not identified during the information review (i.e. resources that could be impacted or existing drainage problems that should be remedied). An example would be a wildlife species habitat that could only be identified from a field investigation.

Data with different levels of detail is required at different steps within the planning and design process. Data is collected, analysed, recorded and stored throughout the highway planning and design process. Data specifics will depend upon the impacts of the highway project, the scale and extent of the highway project, the watershed where the highway is located and the stage in the highway planing and design process that the project has progressed to.

A data program should be developed at the beginning of the planning and design process. The following items should be included in the program:

- data specifications;
- data collection - spatial extent, frequency, duration, method, scale, presentation;
- data recording and storage;
- data interpretation;
- data integration; and
- data reporting.

Chapter 7 in Part 2 of this manual can be used to identify various data sources.

Supporting Studies

Supporting studies may have to be conducted to provide numerical estimates of drainage criteria. As an example, a drainage criterion may have been developed to ensure that all highway crossings must be designed to convey a 25-year flood without overtopping. Calculations and flow measurements may have to be undertaken to determine the peak discharge rate for the 25-year flood. In addition, studies may be undertaken to inventory existing problems such as stream-bank erosion.

Conflict Resolution

There may be a conflict between the drainage objectives and criteria developed by regulatory agencies or the public, and drainage objectives and criteria developed by MTO. As an example, one external agency may have a policy that discourages the use of stormwater infiltration, while another agency may advocate the use of infiltration. Conflict between the criteria of regulatory agencies or the public, could be resolved by applying the conflicting drainage objectives and criteria in an evaluative role (i.e. to compare drainage options) rather than an exclusionary role (i.e. to eliminate drainage options). This would allow tradeoffs to be made and a selection of the drainage option with the least overall impacts.

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Appendix 2A: Possible Drainage Impacts

Table 2A.1: Potential Hydrologic Impacts, Their Causes and Possible Effects

Potential Hydrologic Impacts	Possible Causes	Possible Subsequent Effects
H1. Increases in surface and other rapid runoff: • greater runoff volumes; • greater peak flows; • higher frequency of occurrence of storm runoff events (i.e. greater number of runoff events in the year); • higher frequency of occurrence of extreme storm runoff peak flows (e.g. the former 10 year peak flow might now be expected to occur once in two years on average); • differences in storm hydrograph shape; • differences in the seasonal distribution of storm runoff amounts;	**a) Installation of less permeable ground cover(s).** **Example:** • Installation of a relatively impermeable ground cover such as pavement causes an increase in surface runoff. As a result, the number of runoff events may increase, the volume of storm runoff will increase - in each event and for the year, peak flows are likely to increase, and there is less water available to infiltrate possible recharge groundwater aquifers. **b) Changes in the surface topography or the surface drainage system which expand the land area drained by surface flows.** **Example:** • Cutting and/or filling can alter local watershed boundaries, increasing the surface area draining in a particular direction. • The construction of ditches or waterways can extend the natural drainage system, often resulting in the surface drainage of an increased area of land and more rapid drainage of that land. **c) Changes in the spatial redistribution of snow accumulation, and thereforethe spatial and temporal pattern of snowmelt, as a result of the clearing of vegetation and/or changing surface topography or the surface drainage system.** **Example:** • Snow accumulates in and adjacent to well-vegetated areas. Therefore, the removal and/or planting of vegetative cover may cause a change in the spatial accumulation pattern of snow, and in the temporal and spatial patterns of snowmelt. If this accumulation and melt is near a natural or constructed surface drainage system, the spring surface runoff hydrograph is likely to exhibit a greater volume and a changed shape. • Since snow accumulates in excavated areas, the construction of ditches and other waterways results in an accumulation of snow in these areas and the possibility of increased and more rapid snowmelt runoff	**i) Changes in stream regime** (See Geomorphologic Impacts) **ii) Increased land, streambank and streambed erosion** (See Soil Erosion and Geomorphologic Impacts) **iii) Dilution of some streamflow contaminants** (See Water Quality Impacts) **iv) Increased downstream flooding** (See Socioeconomic Impacts)

Table 2A.1: Potential Hydrologic Impacts, Their Causes and Possible Effects

Potential Hydrologic Impacts	Possible Causes	Possible Subsequent Effects
H2 Decreases in surface or other rapid runoff: • lesser runoff volumes • lesser peak flows • lower frequency of occurrence of storm runoff events • lower frequency of occurrence of extreme runoff peak flows • differences in hydrograph shape • differences in the seasonal distribution of storm runoff amounts	**a) Changes in the surface topography or the surface drainage system which reduces the land area drained by surface flows.** **Example;** • Highway rights-of-way and profiles can disrupt the continuity of natural drainage systems, impounding or limiting surface runoff. **b) Changes in the spatial redistribution of snow accumulation, and therefore the spatial and temporal patterns of snowmelt, as a result of the clearing of vegetation and/or changing surface topography or the surface drainage system.** **Example:** • Cuts and fills, and embankments, may result in changed snow redistribution patterns, as a result of alterations in the air currents around the new topographic features, and the possibility of changed surface runoff patterns, including decreases in surface runoff.	**i) Lack of water for wetlands dependent on surface runoff inputs** (See Terrestrial Biota Impacts) **ii) Lack of water to dilute contaminant loads** (See Water Quality Impacts) **iii) Lack of water to transport sediments** (See Geomorphologic Impacts)
H3 Increases in groundwater levels and runoff: • higher water table levels • more and/or larger wetlands • more groundwater runoff	**a) The trapping or impoundment of surface runoff.** **Example:** • The construction of highway embankments, rights-of-way and profiles which trap and/or impound surface runoff, make more water available to infiltrate the soil and possibly recharge groundwater, in which case local water table levels could rise, associated wetlands could expand and groundwater contributions to nearby streams could increase. **b) Reduction of evapotranspiration.** **Example;** • The removal of plants with roots in shallow groundwater systems reduces evapotranspiration losses from those systems.	**i) Changes in vegetation which is dependent on shallow water table systems.** (See Terrestrial Biota Impacts)

Table 2A.1: Potential Hydraulic Impacts, Their Causes and Possible Effects

Potential Geomorphology	Possible Causes	Possible Subsequent Effects
H4 Decreases in groundwater levels and runoff: • lower local water table levels • fewer and/or smaller wetlands • less groundwater runoff	**a) A reduction in opportunities for water to infiltrate into the soil.** **Example:** • Installation of relatively impermeable ground covers such as pavement reduces the water available to infiltrate to and possibly recharge groundwater systems, resulting in lower local water table levels, fewer and or smaller wetlands and less water available for groundwater contributions to streamflow. • The more rapid removal of surface water by increasing ground slopes and/or constructing more efficient and extensive surface drainage systems reduce the water available for infiltration. **b) The drainage of shallow groundwater systems with the installation of surface and/or subsurface drains.** **Example:** • Cuts, trenches and ditches can intercept and drain shallow groundwater systems, lowering local water table levels, draining nearby wetlands and reducing groundwater discharge.	**i) Mortality or loss of vegetation which is dependent on sustained shallow water table levels.** (See Terrestrial Biota Impacts) **ii) Mortality or loss of aquatic life including fish which are dependent on sustained and cool groundwater flows.** (See Aquatic Biota Impacts)

Table 2A.2: Potential Geomorphologic Impacts, Their Causes and Possible Effects

Potential Geomorphology	Possible Causes	Possible Subsequent Effects
G1 Higher flow velocities: • higher localized flow velocities in general and/or • the more frequent occurrence of high flow velocities	**a) Greater stream flows.** **Example:** • An increased volume of stream flow usually results in increased flow velocities, at least in local reaches of the stream. Therefore, activities leading to increased runoff (see H1 and H3) usually lead to a greater frequency of higher flow velocities. **b) Narrowing of the stream cross-section.** **Example:** • A narrower cross-section at a given stream location tends to exhibit deeper and more rapid flows. Stream crossings or channel modifications which restrict the stream width usually result in a deepening of the stream, leading to increased and more erosive velocities. **c) Reduction in channel roughness.** **Example:** • Stream crossings or channel modifications which make use of stream bank or bed materials which are less hydraulically rough (e.g. concrete, sheet steel) inevitably result in higher flow velocities. • The clearing and grubbing of bank and floodplain vegetation usually leads to smoother hydraulic conditions and higher flow velocities. **d) Steepening of the channel gradient and/or shortening of the channel.** **Example:** • The realignment of a highway in a floodplain often results in relocation of the stream channel, including straightening of the channel and an associated steepening of the channel gradient.	**i) Increased stream bank and/or stream bed erosion.** (See G5) **ii) Increased sediment transport capacity of the stream.** (See G6) **ii) Damage or destruction of the aquatic ecosystem** (See Aquatic Biota Impacts)

Table 2A.2: Potential Geomorphologic Impacts, Their Causes and Possible Effects

Potential Geomorphology	Possible Causes	Possible Subsequent Effects
G2. Lower flow velocity: • lower localized flow velocities in general and/or • the less frequent occurrence of high flow velocities	**a) Smaller stream flows.** **Example:** • Smaller stream flows usually result in lower flow velocities. Therefore, activities leading to decreased runoff (see H2 and H4) usually lead to a lower frequency of higher flow velocities. **b) Widening the stream channel.** **Example:** • Shallower flows at a given cross-section tend to exhibit lower flow velocities. Therefore, stream crossings or channel modifications which widen the channel, passing given flows at shallower depths, often lead to much reduced flow velocities. **c) Increase in channel roughness.** **Example:** • Stream crossings or channel modifications which make use of stream bank or bed materials which are more hydraulically rough (e.g. large rock, groynes) can result in reduced flow velocities.	**i) Reduced sediment transport capacity of the stream.** (See G6) **ii) Flows that are too shallow and/or too slow to sustain or support the aquatic community.** (See Aquatic Biota Impacts).
G3. Deeper flow depths: • deeper localized flow depths in general and/or • the more frequent occurrence of deeper flow depths.	**a) Greater stream flows.** **Example:** • Increased volume of stream flow usually results in increased flow depths. Therefore, activities leading to increased flows (see H1 and H3) usually lead to an increased frequency of deeper flows. **b) Narrowing of the stream cross-section.** (See G1-b)	**i) Increased stream bank and/or stream bed erosion.** (See G5) **ii) Increased local flooding.** (See Terrestrial Biota and Socioeconomic Impacts)
G4 Shallower flow depths: • shallower localized flow depths in general; and/or • the less frequent occurrence of deeper flow depths.	**a) Smaller stream flows.** **Example:** • Activities leading to decreased flows (see H2 and H4) usually lead to an increased frequency of shallower flows. **b) Widening of the stream cross-section.** (See G2-b) **c) Steepening of the channel gradient.** (See G1-d)	**i) Flows too shallow to support and/or sustain the aquatic community.** (See Aquatic Biota Impacts)

Table 2A.2: Potential Geomorphologic Impacts, Their Causes and Possible Effects

Potential Geomorphology	Possible Causes	Possible Subsequent Effects
G5 Increased sediment loads: • increased suspended and/or bed load and/or • increased size of suspended and/or bed material.	**a) Increased supply of sediment.** **Example:** • Activities associated with highway projects which lead to increased soil erosion (See SE1) and increased amounts of soil transported to stream channels (See SE2) usually result in increased sediment loads in those streams. • The use of rock riprap as a soil erosion control measure in surface drains and stream channels provides a potential source of bed material which is much larger in size than most of the natural soil materials. The sediment regime of the waterway or stream is thus dramatically altered, increasing the potential for altering the entire stream regime (See G9). **b) Increased capacity to transport sediment.** **Example:** • Highway undertakings which increase stream flows (See H1), increase stream velocities (See G1), and/or increase channel gradients (See G1-d) result in flow situations with increased stream power and competence. That is, the stream flow is capable of transporting more sediment and sediment involving larger particles.	**i) Damage or destruction of the aquatic biota** (See Aquatic Biota Impacts) **ii) Increased costs to remove sediment at water supply intakes** (See Socioeconomic Impacts)
G6. Decreased sediment loads: • decreased suspended and/or bed load; and/or • decreased size of suspended and/or bed material.	**a) Decreased supply of sediment.** **Example:** • Soil erosion and sediment control measures introduced as part of a highway undertaking (e.g. a sediment detention pond) can result in downstream sediment loads which are less than those which occurred naturally. **b) Decreased capacity to transport sediment.** **Example:** • Highway undertakings which decrease stream flows (See H2), decrease stream velocities (See G2), and/or decrease channel gradients (e.g. by means of flow control weirs) result in flow situations with decreased stream power and competence. That is, the stream flow is capable of transporting less sediment and sediment of smaller size.	**i) A change in the regime or form of the stream.** (See G9)
G7 Degradation of the channel: • increased stream bank erosion and/or • increased stream bed erosion	**a) Greater stream flow volumes and peaks.** (See H1) **b) Higher flow velocities.** (See G1)	**i) Increased sediment loads.** (See G5 and Water Quality Impacts) **ii) Damage or destruction of the riparian ecosystem.**

Table 2A.2: Potential Geomorphologic Impacts, Their Causes and Possible Effects

Potential Geomorphology	Possible Causes	Possible Subsequent Effects
	c)Deeper flow depths. (See G3)	(See Terrestrial Biota Impacts) **iii) Damage or destruction of the aquatic ecosystem.** (See Aquatic Biota Impacts) **iv) Loss of property and/or facilities** (See Socioeconomic Impacts)
G8 Aggradation of the channel: • decreased stream bank erosion • decreased stream bed erosion • deposition of sediments	**a) Lesser stream flow volumes and peaks.** (See H2) **b) Lower flow velocities.** (See G2) **c)Shallower flows.** (See G4)	**i) Decreased sediment loads.** (See G6 and Water Quality Impacts) **ii) Damage or destruction of the aquatic ecosystem.** (See Aquatic Biota Impacts)
G9. A change in the regime or form of the stream: • the width: depth ratio of the cross-section and/or its rate of change • the sinuosity or meander pattern of the stream and/or its rate of change • the bed form(s), e.g. riffles, pools, dunes, and/or their rates of change	**a) A change in the flow regime.** **Example:** • Highway undertakings which lead to a change in the flow regime of the stream, involving one or more of the items identified in H1 and H2, can lead to changes in the stream hydraulics and result in alterations in the fundamental geomorphology of the stream. **b) A change in the sediment regime.** **Example:** • Just as in a) above, highway undertakings which lead to a change in the sediment regime of the stream (See G5, 6, 7 and 8) can also lead to changes in the fundamental geomorphology of the stream.	**i) A change in sediment loads.** (See G5, G6 and Water Quality Impacts) **ii) Loss of or damage to riparian and/or floodplain ecosystems.** (See Terrestrial Biota) **iii) Loss of or damage to riparian and/or floodplain lands and/or facilities** (See Socioeconomic Impacts)

Table 2A.3: Potential Soil Erosion Impacts

Potential Soil Erosion Impacts	Possible Causes	Possible Subsequent Effects
SE1 Increased rates and volumes of soil erosion from the landscape by water	**a) Decreased vegetative cover** **Example:** • Any highway project which involves stripping of the natural vegetative cover leaves the soil surface vulnerable to the erosive forces of rainfall and surface runoff, until such time as that surface is once again protected be it with natural or synthetic products. **b) Increased ground slopes and/or slope lengths** **Example:** • Embankments, rights-of-way and highway profiles which involve the creation of steeper ground slopes and/or the lengthening of slope profiles are likely to cause increased soil erosion, unless and until such slopes are protected. **c) Increased rates and/or volumes of surface runoff** **Example:** • Activities which lead to increased surface runoff (see H1 and H3) often lead to increased soil erosion. Widespread runoff results in increased sheet and rill erosion; concentrations of runoff result in the formation of rills and even gullies.	**i) Loss of productive agricultural land.** (See Socioeconomic Impacts) **ii) Creation of unstable slopes and unsafe gullies.** (See Socioeconomic Impacts) **iii) Undermining of buildings.** (See Socioeconomic Impacts) **iv) Deposition of erosion soil on unique or highly-valued land, e.g. agricultural fields, significant wetlands.** (See Terrestrial Biota and Socioeconomic Impacts)
SE2. Increased amounts of soil transported across the landscape by surface runoff into nearby waterways and streams	**a) Increased soil erosion** (See E1) **b) Increased surface runoff** **Example:** • Activities which lead to increased surface runoff (see H1) provide the transport medium to carry erosion soil to nearby waterways and streams. **c) More extensive and efficient surface drainage systems** **Example:** • Ditches and other waterways constructed to provide effective and efficient surface drainage for highway projects also provide ready conveyance of erosion soil, usually as suspended material, into nearby streams.	**i) Increased stream sediment loads.** (See Geomorphology and Water Quality Impacts) **ii) Damage or destruction to aquatic ecosystems.** (See Aquatic Biota Impacts) **iii) Degradation of the aesthetic quality of the stream.** (See Socioeconomic Impacts) **iv) Increased costs at water supply intake facilities.** (See Socioeconomic Impacts)

Table 2A.4: Potential Water Quality Impacts, Their Causes and Possible Effects

Potential Water Quality Impacts	Possible Causes	Possible Subsequent Effects
WQ1 Changes in the water chemistry of streams and wetlands brought about by new inputs of materials: • toxic chemicals • nutrients • salts	**a) Losses of material from vehicles.** **Example:** • The introduction of a highway into an area or the expansion of an existing highway system results in losses and increased losses of oil/grease, trace organics, trace metals and nutrients from vehicles and vehicle exhausts. • The upgrading of highway systems often results in an increase in the incidence of chemical spills resulting from accidents and of illegal dumping of liquid or solid material (contaminated water may be leached from solid material through the accumulation of precipitation, e.g. mounds of asphalt). **b) Road Maintenance Procedures** **Example:** • Road salting, sanding and road maintenance practices on highways increase the chance of chemicals getting into the adjacent water systems. • Use of pesticides to control roadside vegetation often alters the water chemistry of adjacent streams and wetlands. • There can be movement of materials such as salt from storage sites to local drainage systems.	**i) Changes in animal or plant species or communities** (See Aquatic Biota Impacts) **ii) Contamination of groundwater used for drinking water and the maintenance of base flow in streams** (See Socioeconomic Impacts) **iii) Increased requirements for municipal, industrial or agricultural users of surface or ground water supplies for water treatment** (See Socioeconomic Impacts) **iv) Increases in nutrient loads to downstream aquatic ecosystems** (See WQ2) **v) Contamination of aquatic or surface sediments** (See WQ3)

Table 2A.4: Potential Water Quality Impacts, Their Causes and Possible Effects

Potential Water Quality Impacts	Possible Causes	Possible Subsequent Effects
WQ2. Changes in water quality due to alterations in physical processes only or in combination with the excess growth of aquatic plants: • suspended sediment • temperature • nutrients • organic material • dissolved oxygen • acid-base balance	**a) Changes in erosion rates.** **Example:** • Highway projects which result in increased soil erosion and stream sediment (See Soil Erosion Impacts) can result in associated degraded water quality. • Projects which lead to an increase in bank erosion and associated suspended sediment loads, due to increases in flow velocities and/or the frequency of bank full flow conditions (See Hydrology and Geomorphology Impacts) also result in degraded water quality. **b) Changes in the inputs of energy.** **Example:** • A combination of: 1) geomorphological impacts, causing an increase in stream width and consequently shallower base flows, 2) a reduction in summer base flow volume, and 3) losses of riparian vegetation, can result in increases in tributary water temperatures in the summer and decreased temperatures in the winter. **c)Increased nutrient supplies** **Example:** • An increase in nutrients, coupled with increases in water temperature and light regimes, stimulates the growth of algae and macrophytes and can result in large diurnal fluctuations of pH and dissolved oxygen in stream and some wetland ecosystems. **d) Changes in organic matter quality and supply.** **Example:** • The removal of riparian vegetation alters inputs of organic matter to the adjacent stream and in the ability of the stream to trap organic matter through fallen logs and branches. Such removal therefore leads to possible changes in the quality and quantity of organic material in the stream, and structure of benthic communities. Such changes in turn affect the quantity and quality of food and habitat for fisheries.	**i) Changes in animal or plant species or communities** (See Aquatic Biota Impacts)

Table 2A.4: Potential Water Quality Impacts, Their Causes and Possible Effects

Potential Water Quality Impacts	Possible Causes	Possible Subsequent Effects
WQ3. Changes in aquatic and surface sediment quality	**a) Contaminated Stormwater**. **Example:** • Contamination of suspended sediment with trace metals and/or trace organics found in stormwater can result in aquatic sediments or surface sediments which are hazardous to biota.	**i) Losses or reductions in animal or plant species or communities** (See Aquatic Biota Impacts) **ii) Disruptions in animal behaviour** (See Aquatic Biota Impacts) **iii) Restrictions on downstream dredging** (See Socioeconomic Impacts)
WQ4. Localized contamination of air, dry deposition, or wet deposition	**a) Pollutants associated with vehicle exhaust or construction and maintenance activities** **Example:** • Wet or dry deposition of salty, oily or dusty material from highways onto nearby native plant communities, tree plantations or agricultural crops can cause significant damage. • Drift of pesticides used for roadside vegetative control can damage or destroy non-target plants or insects. The loss of pollinating insects, due to non-target impacts of pesticide use, can also affect some agricultural crops.	**i) Loss of plant species or communities** (See Impacts on Terrestrial Biota or Aquatic Biota) **ii) Contamination or loss of agricultural crops** (See Socioeconomic Impacts) **iii) Contamination of aquatic or surface sediments** (See WQ3)

Table 2A.5: Potential Impacts on Terrestrial Biota, Their Causes and Possible Effects

Potential Impacts on Terrestrial Biota	Possible Causes	Possible Subsequent Effects
TB1. Losses or reductions in native or exotic plant species or communities associated with Terrestrial Ecosystems: • rare and endangered native plant species • forest plantations or agricultural crops	**a) Direct removal or injury of plant cover.** **Example:** • Site preparation and site access for highway projects usually results in the clearing and removal of plant materials. • Plants are trampled and/or their root systems damaged by the use of heavy equipment, or as a result of inadequate buffers, particularly during wet weather conditions when the risk of soil compaction is high. **b) Changes in micro-climate.** **Example:** • The removal of tree canopy during highway undertakings results in local changes in sunlight, soil moisture and temperature regimes **c) Changes in shallow groundwater systems. Example:** • Departures from the normal seasonal fluctuations of the water table are brought about by changes in the surface and groundwater regimes, affecting the survival of existing plants, and providing opportunities for other plants adapted to the new soil moisture regime. **d) Increased soil salinity.** **Example:** • Increases in soil salinity from the loss of highway salt, due to local drainage or spray, affects the growth and survival of salt-sensitive plant species.	**i) Loss of biodiversity.** (See Socioeconomic Impacts) **ii) Changes in the composition of animal species and/or communities.** (See TB3 and TB4) **iii) Changes in the flow of material to Wetland and Tributary Ecosystems.** (See Soil Erosion, Water Quality, Hydrology, Geomorphology and Aquatic Biota Impacts) **iv) Losses of commercial plant species.** (See Socioeconomic Impacts) **v) Expansion in the range of native or exotic plants.** (See TB2)

Table 2A.5: Potential Impacts on Terrestrial Biota, Their Causes and Possible Effects

Potential Impacts on Terrestrial Biota	Possible Causes	Possible Subsequent Effects
TB2. Expansion in the range of native or exotic plant species or communities associated with Terrestrial Ecosystems	**a) Intentional or accidental introduction of exotic plant biota.** **Example:** • The intentional planting of road ditches with non-native species can result in damaging consequences. • Exotic plant species can be introduced through dispersal mechanisms associated with vehicle traffic. • Non-intended weed species are often introduced with the planting of vegetation. **b) (See also TB1 b, c and d).**	**i) Displacement of native plant species.** (See TB1) **ii) Changes in animal species or communities composition.** (See TB3 and TB4) **iii) Changes in the flow of material to Wetland and Tributary Ecosystems** (See Water Quality, and Aquatic Biota Impacts) **iv) Disruptions in the relationships between Terrestrial Ecosystem components** (See TB4)

Table 2A.5: Potential Impacts on Terrestrial Biota, Their Causes and Possible Effects

Potential Impacts on Terrestrial Biota	Possible Causes	Possible Subsequent Effects
TB3. Losses of animal species or communities associated with Terrestrial Ecosystems: • mammals • birds • amphibians • reptiles	**a) Barriers to migration or animal movement:** **Example:** • Highway drainage features may prevent amphibians, reptiles and/or mammals from crossing the route. As a consequence, the animals may not be able to successfully complete their life stages and/or be unable to recolonize habitats where animal populations have been lost due to stresses from natural or anthropogenic sources. Extirpation and/or extinction of some populations may result. **b) Loss of habitat features required by animals:** **Example:** • The removal or destruction of vegetative cover may lead to the loss of particular plants and/or species of plants required by animals for food, nesting habitat or protective vegetative cover. Deer for instance, in an activity referred to as yarding, concentrate their numbers and overwinter in small areas that meet their specific needs for shelter, food and safety. • Changes in the size of available habitat and in the connections to adjacent habitats affects the long term viability of animal populations.	**i) Increased road-kill of animals.** (See TB5 and Socioeconomic Impacts) **ii) Loss of Biodiversity**. (See Socioeconomic Impacts) **iii) Expansion in the range of some terrestrial animals**. (See TB4)
TB4. Expansion in the ranges of animal species or communities associated with Terrestrial Ecosystems: • White-Tailed Deer • Canada Goose	**a) Loss of predators.** **Example**: • The loss of raptors, snakes and other predators due to either toxic chemicals or habitat loss can increase the abundance of some small mammals. **b) Increased habitat.** **Example:** • Forested areas converted to grasses may provide new habitat for some birds or small animals.	**i) See TB5.**
TB5. Disruptions in the relationships between different components of Terrestrial Ecosystems:	**a) Planted roadside vegetation.** **Example:**	**a) Increased road-kill of animals.** (See Socioeconomic Impacts)

Table 2A.5: Potential Impacts on Terrestrial Biota, Their Causes and Possible Effects

Potential Impacts on Terrestrial Biota	Possible Causes	Possible Subsequent Effects
Ecosystems: • foraging patterns of animals	• Changes in the micro-climate, particularly in forested regions, often make these areas the first to produce new plant growth in the spring. Deer and other herbivores are often attracted to this new growth, particularly when palatable plants species have been introduced.	

Table 2A.6: Potential Impacts on Aquatic Biota, Their Causes and Possible Effects

Potential Impacts on Aquatic Biota	Possible Causes	Possible Subsequent Effects
AB1 Losses or reductions in plant species or communities associated with Tributary or Wetland Ecosystems: • riparian vegetation • stream macrophytes • wetland plant species	**a) Direct removal or injury of plant cover.** **Example:** • The filling in or draining of wetlands or riparian areas due to cut and fill operations, borrow pit creation, or the storage of construction material often leads to the removal and/or injury of plant cover. • Downstream aquatic plants are damaged or destroyed, and aquatic habitats are altered, by deposits of sediment lost during construction. • See also TB1, a, b, c, and d.	**i) Loss of plant biodiversity.** (See Socioeconomic Impacts) **ii) Changes in animal species or communities composition.** (See AB3 and AB4) **iii) Increased stream bank erosion.** (See Water Quality and Geomorphology Impacts) **iv) Expansion in the range of native or exotic plants.** (See AB2) **v) Changes in the flow of material to downstream reservoir and lake ecosystems.**
AB2. Expansion in the range of native or exotic plant species or communities associated with Tributary or Wetland Ecosystems: • Cladophera algae • Purple loosestrife	**a) Increased supplies of nutrients.** **Example:** • An increased ambient concentration of phosphorus, transported from highways to tributary and wetland ecosystems, can lead to the increased growth of aquatic plants such as algae and macrophytes. **b) Loss of sensitive native species.** (See AB1) **c) Drainage works**. **Example:** • The drainage of wetland ecosystems may inadvertently cause the dispersal of exotic plants.	**i) Displacement of native plant species.** (See AB1) **ii) Changes in the composition of animal species or communities.** (See AB3 and AB4) **iii) Diurnal fluctuations in the pH and dissolved oxygen concentrations in streams and wetlands.** (See Water Quality Impacts) **iv) Changes in the flow of material to downstream reservoir and lake ecosystems.**

Table 2A.6: Potential Impacts on Aquatic Biota, Their Causes and Possible Effects

Potential Impacts on Aquatic Biota	Possible Causes	Possible Subsequent Effects
AB3 Losses or reductions in animal species or communities associated with Tributary or Wetland ecosystems: • fish • amphibians • reptiles • invertebrates (i.e. crayfish, mollusca, may flies, etc.) • birds • mammals	**a) Acute Stormwater toxicity.** **Example:** • Stormwater discharged directly to tributary and/or wetland ecosystems can be acutely toxic to aquatic biota, even after dilution, due to the high concentration of a single compound or a mixture of organic chemicals and/or trace metals. These compounds often occur in conjunction with high water temperatures, high suspended solids concentrations or low dissolved oxygen concentrations which tend to increase stormwater toxicity. **b) Chronic or non-lethal effects associated with stormwater discharges or with the contamination of aquatic sediments.** **Example:** • Contamination of stormwater and aquatic sediments can result in population, individual, and/or cellular/subcellular effects on stream and wetland biota. These effects can result in: alterations in growth, reproductive success, and/or developmental toxicity, and the bio-magnification of trace organics which can result in the contamination of fish, fish-eating birds, or fish-eating mammals. Demographic changes such as shifts in species composition, distribution, population, biomass and behaviour can also occur. **c) Loss of habitat features required by animals.** **Examples:** • Stream channelization eliminates habitat for benthic animals associated with stream sediments and stream macrophytes and subsequently reduces food availability for fish or birds that feed on those benthic organisms or for birds that feed on emergent insects. • Hydrologic changes can result in increased flows that can scour stream beds and remove invertebrate species and communities associated with fine-grained sediments in tributary ecosystems.	**i) Loss of biodiversity.** (See Socioeconomic Impacts) **ii) Loss of commercial fish species.** (See Socioeconomic Impacts) **iii) Loss of recreational opportunities.** (See Socioeconomic Impacts) **iv) Expanded range of some native or exotic animal species.** (See TB4)

Table 2A.6: Potential Impacts on Aquatic Biota, Their Causes and Possible Effects

Potential Impacts on Aquatic Biota	Possible Causes	Possible Subsequent Effects
	• Changes in stream geomorphology that result in high flow velocities can prevent the upstream migration of fish or other aquatic biota leaving this stream habitat under utilized and reducing the range and resilience of effected animal species and communities. • Low velocities in combination with shallow flows due to stream widening and base flow reductions can have a similar effect by reducing the ability of aquatic biota to migrate or to escape stresses such as high water temperatures. • Disruptions in nutrient cycling (i.e. phosphorus, nitrogen, carbon, trace nutrients) and decomposition rates can either increase or decrease the food supply for animals. • Removal of riparian vegetation can change stream water temperature and light regimes. Such alterations can change in-stream primary productivity, provided the nutrients are available, by stimulating the growth of algae. Stream biota which cannot adapt to the increases in water temperatures or habitat changes will be replaced by more tolerant native or exotic species.	
AB4. Expansion in the range of animal species associated with Wetland or Tributary Ecosystems: • fish	**a) Degraded habitat.** **Example:** • Reductions in the quality of existing habitat provide opportunities for more tolerant fish species.	**i) Loss of animal biodiversity** (See Socioeconomic Impacts)

Table 2A.6: Potential Impacts on Aquatic Biota, Their Causes and Possible Effects

Potential Impacts on Aquatic Biota	Possible Causes	Possible Subsequent Effects
AB5 Disruptions in the relationships between different components of Wetland or Tributary Ecosystems: • foraging patterns of animals, (Canada Geese, birds, etc)	**a) Changes in plant species and communities.** **Example:** • Seasonal availability and quality of food affects the migration and feeding habitats of biota. Canada Geese, for example, have altered their migration and feeding habitats due to changes in plant cover that result from urban and agricultural land-uses. **b) Creation of artificial salt licks.** **Example:** • To meet a dietary need for salt some large mammals such as deer may be attracted to wetlands that accumulate salt due to its winter time use as a highway deicer.	

Table 2A.7: Potential Socioeconomic Impacts, Their Causes and Possible Effects

Potential Socioeconomic Impacts	Possible Causes	Possible Subsequent Effects
S1 Loss of life and/or property.	**a) Hydrology or geomorphology changes.** **Example:** • Increases in high flows as a result of highway work can lead to the flooding of homes, businesses and/or roads, and associated loss of life. • Vehicle collisions with wildlife can result from animals being moved onto the highway system from flooded areas.	**i) Loss of property and/or injuries.**
S2 Loss of agricultural resources. • orchards • cereal crops	**a) Phytotoxicity of wet deposition, dry deposition or air.** **Example:** • Ozone, salt and particulate matter can injure or impair the growth of plants. **b) Alterations in drainage.** **Example:** • Changes to drainage which affect shallow groundwater levels may result in the seasonal flooding and losses of agricultural crops.	**i) Loss of agricultural income.**
S3. Loss of archeological and historic resources.	**a) Loss of or damage to sites of cultural or historic importance to native and non-native peoples.** **Example:** • Cemeteries or burial grounds are often moved. • Historic sites associated with native or non-native settlements are also often altered by drainage or drainage works.	**i) Reductions in the quality of life.**
S4 Increased costs of water treatment: • municipal • industrial • agricultural • domestic	**a) Changes in water quality.** **Example:** • Users of tributary waters which experience increased suspended loads, or increases in dissolved ion concentrations due to changes in water salinity, may be required to increase their treatment of water prior to use. (See also Water Quality Impacts) • Ground water wells contaminated as a result of road salting practices or by other contaminants may need to switch to other water supplies.	**i) Additional cost associated with obtaining an alternate water supply.**

Table 2A.7: Potential Socioeconomic Impacts, Their Causes and Possible Effects

Potential Socioeconomic Impacts	Possible Causes	Possible Subsequent Effects
S5 Loss of beneficial or recreational uses of terrestrial or aquatic biota: • bird watching • fishing • tourism • hiking	**a) Losses of fish or bird species.** **Example:** • Losses occur as a result of the removal or degradation of habitat features. Losses in fishery resources affect the availability of a food supply for both native and non-native fishermen. • Losses in amounts and types of terrestrial and aquatic habitats affect the diversity of bird populations and reduce the opportunity for recreational activities such as bird watching.	**i) Reductions in the quality of life.**
S6 Loss of aesthetics: • objectionable odours • increased noise • decreased satisfaction with visual appearance of landscape	**a) Loss of terrestrial or aquatic plants.** **Example:** • The removal of mature plants such as trees removes an effective barrier to the migration of noise from highways in addition to reducing the quality of the landscape's visual appearance. • Objectionable odours in the summer from streams and wetlands can be the result of an increased growth in algae due to increases in nutrient, light and water temperature.	**i) Reductions in the quality of life.**

Table 2A.7 Potential Socioeconomic Impacts, Their Causes and Possible Effects

Potential Hydrologic Impacts	Possible Causes	Possible Subsequent Effects
S7. Loss of biodiversity.	**a) Losses or reductions in animal or plant species.** (See Aquatic and Terrestrial Biota Impacts and Possible Causes)	**i) Reduction in life-sustaining services such as food and oxygen production, water purification and climate moderation.** **ii) Losses in the biological resource base for pharmaceutical, biotechnological, agricultural, fishing, and forest industries.**

Appendix 2B: Common Law Principles

Note: Appendix 2B is intended to provide information and guidance on the more important legal aspects of highway drainage. For instance, the practitioner needs to have sufficient knowledge of drainage law to be able to recognize and avoid potential legal problems, such as those commonly caused by flow diversions, concentration of flow, obstruction of flows by bridges or culverts, and stream bank erosion. This appendix should not be used as a substitute for legal counsel. Since the legal aspects of highway drainage can be confusing and complex, the advice of MTO legal counsel should be obtained for drainage matters, as necessary.

Natural Watercourses

Almost all the laws governing natural watercourses are founded on the maxim *Aqua currit et debet currere*, i.e. water flows naturally and should be permitted thus to flow[1].

> *The Courts have said, that to constitute a natural watercourse, the channel bank formed by the flowing of water must present to the eye on casual examination the unmistakable evidence of the frequent action of running water. On another occasion that a watercourse is constituted if there is sufficient natural and accustomed flow of water to form and maintain a distinct and defined channel. It is not essential that the supply should be continuous or form a perennial living source. It is enough if the water rises periodically and reaches a fairly defined channel of permanent character. A natural watercourse does not cease to be such if at a certain point it spreads out over a level area and flows for a distance without defined banks before flowing again in a defined channel. Often it is the valley through which the stream runs, and not its low level or low water channel, which is the watercourse*[2].

Riparian Rights and Obligations

A riparian owner is one whose land is in actual contact with a natural watercourse. As such, he has the unique right to drain that land into the watercourse. Where a highway crosses a natural watercourse, the Crown, as owner of the land, acquires riparian rights, and may therefore drain the highway into the watercourse.

[1] *Common Law Aspects of Water* by R.A.W. Irwin and published by Ontario Ministry of Agriculture, Food and Rural Affairs, 1974.
[2] *Drainage Law* by A.B. McIlmoyle and published by The Municipal World, April 1969.

A riparian owner is not only entitled to have water in a natural watercourse flow to his land in its natural state as a benefit, but is also obliged to receive it even if it becomes a nuisance due to flooding, erosion or other reasons. However, the strict rights of riparian owners are tempered by obligations under drainage law and nuisance law; and, the obligation to ensure a sufficient outlet may have more legal force than the riparian owner’s property based right to drain his/her land into the watercourses. On the other hand, persons not riparian owners who obtain an outlet to the stream are liable to a downstream riparian owner whose land is damaged by the increased amount of water. It should also be noted that statutory rights of outlet, such as those under the Drainage Act, in no way interferes with the common law rights of a riparian owner.

Reasonable use of a stream has been defined as a use up to the capacity of the banks of the stream. Determination of the "banks" depends upon the water level selected and has not been explicitly defined in law. The natural banks may be delineated by normal summer flow, an average annual flood having approximately a 2.3 year return period, or a higher flow caused by more severe flood conditions, spring tides or other natural phenomena. In instances of dispute, legal action may be necessary to establish the location of natural banks.

The right to discharge water into a natural watercourse is subject to certain restrictions.

- The riparian owner may not bring in water which has not fallen within the natural watershed. In other words, water from one watershed may not be diverted into another.
- The owner may not assign or sell his/her rights to drain into that watercourse. In essence, this means that to secure riparian rights one must obtain ownership of the land itself. The common law has been modified by s.27 of the Ontario Water Resources Act which provides that *A right or interest in, over, above, upon, across, along, through, under or affecting any land ... in respect of water or sewage works* may be granted to either the Crown or a municipality not withstanding that the right is not appurtenant or annexed to any land of the Crown or municipality. Accordingly, the right to drain into a watercourse may be granted to the Crown or a municipality. However, it is not clear if the full rights of a riparian owner can be transferred (even to the Crown or a municipality) unless the land itself is transferred.
- The right to discharge water into a natural watercourse is also subject to an implied proviso that the riparian owner must be making a “reasonable use” of his/her property.

Use of Water

A riparian owner has the right to have the water flow to him/her in its natural state with regard to both quantity and quality, subject to certain qualifications, and may put the water from the natural watercourse to any reasonable use. This may include irrigation and the watering of livestock. Extraordinary use, such as for industrial purposes, would not be reasonable unless the water were returned to the natural watercourse before it left the user's land, substantially unaltered in quantity (i.e. less that which is absorbed) and quality.

Section 34(3) of the Ontario Water Resources Act restricts the removal of water to 50,000 litres/day. For a greater amount, a permit is required from the Ministry of Environment and Energy (MOEE). Section 34(4) states that if the proposed uses interfere with domestic uses of another, the MOEE may prohibit the taking of such water. A contractor must adhere to both these sections.

Interference with Natural Watercourses

There are many types of works that have been held by the courts to constitute interference with a natural watercourse; these include deepening and widening, removing silt or gravel deposits, channel straightening, the construction of bridges, culverts or stream diversions, and channel maintenance.

It is important to remember that it is the duty of anyone who interferes with a natural watercourse to see that the works are adequate to carry the flow of water, even that resulting from an extraordinary rainfall. If not, he must accept all liability for his/her action and must prove his/her innocence. This is discussed further in the Watercourse Crossing section.

When a natural watercourse becomes silted up or choked by vegetation, there is no liability or obligation upon the owner either to clear the channel or to compensate adjoining landowners for flood damage. However, if he takes it upon himself/herself to clean the channel, he/she is liable for any damage his/her interference may cause[3].

In such circumstances, it is recognized that MTO probably has a duty to clear up its culverts; and, this duty is part of the broader duty to maintain the highway in a good state of repair. In a case where a culvert has been clogged for a lengthy period of time, downstream owners may have developed their properties in such a way that the consequence of a culvert cleanup may be that such downstream owners experience flooding problems. In this situation, legal advice should normally be obtained prior to commencement of the cleanup, whenever practical.

Diversions

A natural watercourse may be diverted by a landowner provided it is returned to its natural location within the same property. However, the riparian owner may be liable for damage due to the diversion, both upstream and downstream, whether or not his/her use of the stream is reasonable. This fact must be considered when MTO constructs diversions for bridge and culvert installations. Normally such diversions are undertaken by permit from the Ministry of Natural Resources (MNR)
under the provisions of the Lakes and Rivers Improvement Act. The Act does not bind the MTO,

[3] *Drainage and the Law* by H. McDougall and published by Civic, March 1976.

but in practice MTO keeps MNR informed of all instances of major diversions, and solicits their comments. MTO, however, is bound by the Environmental Assessment Act, and is required to submit to the MOEE an environmental assessment report whenever a permanent environmentally significant watercourse diversion is proposed in lieu of bridges or culverts. Requirements under the Environmental Assessment Act are discussed further in the Statute Law section.

As mentioned earlier, water may not be diverted from one watershed to another. The courts have held that work which directed water from point A to point C (instead of the natural direction from A to B) is in violation of common law, regardless of how minor the change in drainage pattern may be. Claims might be based on a diminution of flow between A and B, or on an increased flow between points A, C and B. In the upper reaches of a drainage basin each gentle undulation in the topography may define a distinct subwatershed in the eyes of the court.

Where a highway traverses rugged terrain on alternating cuts and fills, diversions of minor amounts of water from one watershed to another may be justified if substantial cost savings can be achieved and if future claims or adverse environmental impacts are unlikely. In any case the affected municipality and conservation authority or Ontario Ministry of Natural Resources should be consulted regarding a change in drainage areas.

Watercourse Crossings

Although common law requires that works substituted for a natural watercourse must accommodate a flow resulting from an extraordinary rainfall, the latter term has not yet been defined. In recognition of accepted engineering practice and economic realities, it is the policy of MTO to design most drainage facilities on the basis of pre-selected storm or flood frequencies.

Road fills crossing natural watercourses may behave as dams and levees if they constrict the flood plain, thereby temporarily increasing flooding upstream. This may provide a basis for suit under common law when it can be shown that significant damage is caused.

Surface Flow

The principles which apply to natural watercourses are different from those for surface flow (i.e. sheet flow), for which a separate and distinct set of common law rules governs the rights and obligations of owners.

Obstruction of Surface Flow

An owner of land, which is at a lower elevation than a neighbouring land, has the right to either allow water from higher land to flow over his/her land, or keep such water off his/her property by dams and banks. The owner should ensure that the dams or banks do not result in unreasonable interference with adjacent property owners' enjoyment of his/her property.

Increase of Surface Flow

An owner who paves the surface of his/her land and thereby increases the rate of surface runoff is not normally liable under common law, as long as the surfacing does not result in unreasonable interference with adjacent property owners' right to enjoyment.

Collection of Surface Flow

If a ditch, pipe or curb and gutter is constructed to collect surface water, it is then necessary to provide a sufficient outlet for the collected water, as no owner has the right to collect surface water in this fashion and discharge it onto the lands of others. Sufficient outlet is defined in Section 1(29) of the Drainage Act as a point at which water can be discharged safely so that it will do no damage to lands or roads. Although not judicially resolved in common law, for the purposes of MTO the definition may also be applied to situations other than those related to the Drainage Act.

MTO should carry collected surface drainage to a sufficient outlet or employ some other solution, such as compensation, that is acceptable to all parties concerned.

Surface Flow and the MTO

Where a roadway embankment intercepts surface flow, MTO is within its legal rights to allow that water to pond behind the embankment. However, if this is likely to cause an adverse environmental impact, such as crop damage, the Environmental Assessment Act would apply. If significant damage to upstream properties is likely, it is good policy to collect and carry this water to a sufficient outlet, even though MTO may not be liable for upstream damage.

On the downstream side of the embankment the use of roadside ditches to intercept surface water should be minimized, unless there are circumstances that may cause undue inconvenience or hardship to the adjacent landowner.

Subsurface Flow

Underground Water in a Defined Channel

Insofar as the rights and obligations of landowners are concerned, subterranean flowing streams that have definite courses may be treated for all practical purposes, as natural watercourses on the surface[4]. Thus an owner is entitled to put an underground stream to any reasonable use.

[4] *Gale on Easements* by S.G. Maurice and published by Sweet & Maxwell Ltd., London, England, 1972.

Underground Water not in a Defined Channel

Historically, common law in Ontario has upheld the right of landowners to put underground water to whatever use they want, regardless of the effect on their neighbours' supply. However, in the light of growing concern for the environment, future claims for negligence and nuisance caused by indiscriminate interference with ground water supplied may meet with success in the courts. Therefore, the designer should consider the effect his/her proposal may have on the underground water system and, where potential impacts are significant, incorporate mitigating measures into the design and into the appropriate Environmental Assessment Report.

The common law rules applicable to the obstruction or collection of underground flow not in a defined channel, or percolation, are the same as for surface flow. Thus it is necessary for collected subsurface water to be taken to a sufficient outlet.

Appendix 2C: Statute Law

Note: Appendix 2C is intended to provide information and guidance on the more important legal aspects of highway drainage. For instance, the practitioner needs to have sufficient knowledge of drainage law to be able to recognize and avoid potential legal problems, such as those commonly caused by flow diversions, concentration of flow, obstruction of flows by bridges or culverts and stream bank erosion. This appendix discusses each of the statutes shown on Table 2.4. All text shown in italics is a direct quote from the relevant legislation. This appendix should not be used as a substitute for legal counsel. Since the legal aspects of highway drainage can be confusing and complex, the advice of MTO legal counsel should be obtained for drainage matters, as necessary.

Canadian Environmental Assessment Act, R.S.C., C-37

The Act establishes a process for the environmental assessment of projects that involve the federal government. The Act binds Her Majesty in right of Canada or a province when any project has federal involvement. Included would be the cost sharing of roads or bridges. The Act replaces the Environmental Assessment Review Process (EARP) Guidelines Order and is administered by the Minister of the Environment through the Canadian Environmental Assessment Agency. The Act applies to projects if a federal authority has any of the following involvement:

- is the proponent;
- provides financial assistance;
- administers the land required for the project;
- issues a permit or license for the project.

The Act applies to a federal Minister of the Crown, an agency or other body of the federal government that is accountable to Parliament, and any federal department or departmental corporation. Project means:

(a) in relation to a physical work, any proposed construction, operation, modification, decommissioning, abandonment or other undertaking in relation to that physical work, or

(b) any proposed physical activity not relating to a physical work that is prescribed or is within a class of physical activities that is prescribed by 59(b). Projects include timber cutting in a national park and ocean dumping.

The Act ensures that:

(a) the environmental effects of projects receive careful consideration;
(b) projects do not cause significant adverse environmental effects; and
(c) there is an opportunity for public participation in the assessment process.

Also, the Act encourages authorities to promote sustainable development to achieve or maintain a healthy environment and economy.

Environmental assessment falls into three categories: (1) screening; (2) comprehensive study; and (3) mediation panel review.

Fisheries Act, R.S.C., 1985, F-14

The Fisheries Act is administered by the Federal Department of Fisheries and Oceans. The Act deals with fishery leases/licenses, lobster fisheries, construction of fishways, general prohibitions regarding fish catches and provisions for fish habitat protection and pollution prevention. The Act binds MTO, other provinces and ministries within the Federal government. The Ontario Ministry of Natural Resources administers and enforces the sections of the Fisheries Act regarding habitat.

Section 35(1) of the Fisheries Act states that:

> *No person shall carry on any work or undertaking that results in the harmful alteration, disruption of destruction of fish habitat.*

The exception is with the permission of the Federal Minister of Fisheries and Oceans. *Fish habitat means spawning grounds and nursery, rearing, food supply and migration areas on which fish depend directly or indirectly in order to carry out their life processes.*

Section 35(3)

> *....... no person shall deposit or permit the deposit of a deleterious substance of any type in water frequented by fish or in any place under any conditions where the deleterious substance or any other deleterious substance that results from the deposit of the deleterious substance may enter any such water.*

The exception is with the permission of the Federal Minister of Fisheries and Oceans.

Deposit means any discharging, spraying, releasing, spilling, leaking, seeping, pouring, emitting, emptying, throwing, dumping or placing.

Deleterious means:

(a) any substance that, if added to any water, would degrade or alter or form part of a process of degradation or alteration of the quality of that water so that it is rendered or is likely to be rendered deleterious to fish or fish habitat or to the use by man of fish that frequent the water; or

(b) any water that contains a substance in such quantity or concentration, or that has been so treated, processed or changed, by heat or other means, from a natural state that it would, if added to any other water, degrade or alter or form part of a process of degradation or alteration of the quality of that water so that it is rendered or is likely to be rendered deleterious to fish or fish habitat or to the use by man of fish that frequent that water.......

Deposit includes the discharge of stormwater and deleterious substances include sediment and stormwater. MTO must get approval for stormwater management works that discharge to fish habitat and must get approval for in-stream works that could affect fish habitat.

Navigable Waters Protection Act, R.S.C. 1985, N-22

The Navigable Waters Protection Act is federal legislation "respecting the protection of navigable waters." MTO is bound by the Navigable Waters Protection Act.

A *"navigable water" includes a canal and any other body of water created or altered as a result of the construction of any work* (S.2). Work that interferes substantially with navigation must be approved by the Minister of Transport, as must all bridges, booms, dams or causeways on navigable waters (S.5(2)).

Section 22 of the Act restricts dumping in navigable waters. Any material such as stone, gravel or earth, which may sink, may not be dumped into a navigable water where there are not at least twenty fathoms (36. 6 m) of water at all times.

In view of these restrictions, MTO staff should contact Transport Canada if a construction program will entail building a bridge or culvert over, or dumping material into, a navigable water. The procedure for obtaining approval under this Act is rather complex, and further information may be obtained from Transport Canada's Application Guide to the Navigable Waters Protection Act [5]. Whether or not a particular water is navigable is a matter of fact, and must be decided by Transport Canada through the Coast Guard with respect to each case.

It is important to recognize that a requirement for a Navigable Waters Protection Act permit is one of the environmental assessment triggers alluded to under the Canadian Environmental Assessment Act.

[5] *Application Guide, Navigable Waters Protection* by Transport Canada

Bridges Act, R.S.O. 1990, B.12

The Act applies to rivers or streams where the bed is vested to Her Majesty in right of Ontario or where Her Majesty in right of Ontario is a riparian owner.

Section 2(1) states:

No bridge or other structure shall be built, placed or constructed over or across any river or stream or part thereof, nor shall any bridge or other structure over or across any river or stream or part thereof be rebuilt, replaced or altered, where the cost of such building, placing, constructing, rebuilding replacing or altering will exceed $2,000, except with the approval of the Lieutenant Governor in Council.

The Lieutenant Governor may approve a bridge upon receipt of a request for approval, proof that the plan has been deposited with the Minister of Transportation and proof that the application has been published in the Ontario Gazette and two local newspapers.

Environmental Assessment Act [6], R.S.O. 1990, E.18

Section 2 of the Act states that *"the purpose of this Act is the betterment of the people of the whole or any part of Ontario by providing for the protection, conservation and wise management in Ontario of the environment."* Section 4 states that the Act binds MTO.

Under the terms of the Act, each proponent of an undertaking must submit for approval by the Ministry of Environment and Energy (MOEE) an environmental assessment of the proposed undertaking. The Provincial Highways Class Environmental Assessment defines circumstances under which some MTO projects are pre-approved. Such projects may include improvements to existing highway, stream crossings, watercourse alterations, maintenance and operation improvements.

The environmental assessment should provide the purpose of the undertaking, a description of the rationale used in development of the undertaking, and include an analysis of the effects on the environment of the undertaking, alternative methods of carrying out the undertaking, alternatives to the undertaking, and measures to reduce the impact on the environment. Generally, the preferred scheme is the one having the least disruptive effect on the environment.

Where an undertaking is subject to, and has not received, approval under the Act, no agreement, license or permit can be signed or issued by MTO until such approval has been obtained from the Minister of the Environment and Energy.

[6] This Act has been amended. Refer to the official statute for details.

Environmental Protection Act, R.S.O. 1990, E.19

Section 3 states that the purpose of the Environmental Protection Act is to provide for the protection and conservation of the natural environment. MTO is bound by the Environmental Protection Act.

Section 14 *states no person shall discharge a contaminant into the natural environment that causes or is likely to cause an adverse effect that includes the following:*

(a) impairment of the quality of the natural environment for any use that can be made of it,
(b) injury or damage to property, to plant or animal life,
(c) harm or material discomfort to any person,
(d) an adverse affect on the health of any person,
(e) impairment of the safety of any person,
(f) rendering any property or plant or animal life unfit for human use,
(g) loss of enjoyment of normal use of property, and
(h) interference with the normal conduct of business.

Consideration of potential impacts on the natural environment should be made during the planning, design, construction, operation and maintenance of stormwater management works. MTO staff should ensure that stormwater management works conform with the provisions of the Act, since it binds MTO.

Interpretation Act, R.S.O. 1990, I.11

MTO staff should be aware that in general no Act of the Ontario Legislature binds the Crown, as represented by the Minister of Transportation in the case of MTO, unless it is expressly stated herein that the Crown is bound by that Act.

Section 11

No Act affects the rights of Her Majesty, Her heirs or successors, unless it is expressly stated therein that Her Majesty is bound thereby.

An example of an Act binding MTO is the Environmental Assessment Act, and one not binding MTO is the Tile Drainage Act. Where there is any doubt as to the applicability of an Act to MTO, advice from MTO's legal staff should be obtained. The federal Interpretation Act contains a similar provision.

Section 91 and 92 of the Constitution Act, 1982 delimit the areas of the federal and provincial governments' respective jurisdictions to legislate. Each level of government is intended to have exclusive legislative competence over the subject matters assigned to it by the constitution; but

some areas of overlap do exist. Subject to certain exceptions, the general rule is that no statute will bind the Crown unless express language to this effect has been employed; and, as a matter of statutory construction, a provision binding the Crown will be interpreted to apply only to the legislating government unless there is clear language to show the Crown in right of other jurisdictions is also to be bound.

Municipalities may enact by-laws and ordinances only with strict confines of the jurisdiction expressly conferred upon them by the Province. Given that no provision allowing municipalities to bind the Crown is contained within the Municipal Act, neither federal nor provincial agencies are bound by laws made at the municipal level. Notwithstanding this Crown immunity, however, it is generally the policy of the provincial government to behave as if bound by municipal laws to the extent possible.

Limitations Act, R.S.O. 1990, L.15

There are certain rights that are acquired with land ownership, namely:

- the right to lease the land;
- the right to use the land;
- the right to give the land away;
- the right to enter and restrict, entry onto the land;
- the right to refrain from any activity; and
- the right to and sell the property.

This is known as the **bundle of rights**. The most common title to property is called **fee simple**, in which the landowner receives all of the above rights. There are different means by which these rights may be taken away or restricted, one of which is the Limitations Act. The Act allows a party who has used land for a long period of time without ownership to continue that use.

Prescriptive Rights

Where the rights to the use of another person's property have been established over an extensive period of time, they are known as **prescriptive rights**. In addition to the rights having been established over many years, the use must have been continuous, open and adverse, **Continuous** implies that the use has not been disrupted during the entire period of time. **Open** indicates that the use is not secret, and **adverse** means that the use is against the interest of the owner. The use cannot be adverse if it is with the owner's consent, in the form of an agreement, deed, or permit.

The burden of proof of any claim to prescriptive rights lies with the claimant and not with the registered owner. MTO, like any private individual, can secure prescriptive rights against the lands of others.

The above discussion of prescriptive rights is qualified by noting that there are two systems of recording the ownership of land in Ontario: the Land Titles System and the Registry System. Pursuant to s.51 of the Land Titles Act, no right or interest in land registered under that Act can be acquired by any length of possession or prescription. Under the Limitations Act, which would apply to lands held in the Registry System, adverse possession may be established by showing 20 years of uninterrupted use; however, where Crown land is involved the period of uninterrupted use must be 60 years.

Reviewing a Claim to Prescriptive Rights

The preceding discussion indicates that the establishment of prescriptive rights on highway rights-of-way is rare and thus, ordinarily, there is no legal requirement to accommodate private drainage facilities on MTO land.

The following are some of the questions to be asked in proving or refuting a claim of prescriptive rights:

- When was the highway designated?
- How long has the land been the property of the Crown?
- Is the claim or part thereof on an unopened road allowance?
- How long has the claimant enjoyed the right of uninterrupted use of the land?
- What is the nature and extent of the claimant's use of land?
- Is the use continuous, open and adverse?
- Is the use authorized by a deed, agreement or permit?

The above discussion of a complicated aspect of the law has been purposely simplified, and it is essential that legal counsel be involved in any matter concerning the Limitations Act.

Public Transportation and Highway Improvement Act, R.S.O. 1990, P.50

The Public Transportation and Highway Improvement Act, administered by MTO, contains certain provisions relevant to drainage.

Drainage of Provincial Highways

Section 26 states:

> *The Minister may construct, extend, alter, maintain and operate such works as he or she considers necessary or expedient for the purposes of the Ministry and the Minister and any person, including a municipality or local board thereof, may enter into agreements, with respect to the construction, extension, alteration, maintenance or operation of such works.*

Section 26 authorizes the construction by the Minister of whatever drainage works the Minister deems necessary or expedient.

The Minister is also authorized, under Section 25(1), to initiate proceedings under other Acts in order to procure drainage works. This allows the Minister to use the Drainage Act petition procedure.

Encroachment Permits

Permission for drainage works, other than those of MTO, to be constructed on provincial highways, other than by MTO, may be granted by means of encroachment permits. MTO may specify such conditions as it deems necessary for the granting of permits.

To avoid delays, it is necessary that applicants apply well in advance of advertising a drainage contract for tenders. Application forms should be obtained from the appropriate MTO District Office.

Ontario Water Resources Act, R.S.O. 1990, O.40

The Ontario Water Resources Act is administered by the Ontario Ministry of Environment and Energy (MOEE) and the Act binds the Crown. Two sections of the Act are of particular importance to MTO.

- Section 30 provides that any person who causes or permits the discharge of any material which may impair the quality of the water is guilty of an offence. Accordingly, in constructing or maintaining drainage works, MTO staff or contractors hired by MTO should take appropriate precautions to avoid committing an offence. There is also a duty to report such a discharge should a discharge occur.
- Section 53 of the Act creates a requirement to obtain a Certificate of Approval for a “sewage works”. The term “sewage works” is defined broadly and would include a system for the transmission of highway stormwater. However, clause 53(6)(e) OWRA creates an exception from the requirement to obtain a Certificate of Approval for drainage works constructed under either the Drainage Act or the PTHIA. Accordingly, in most cases MTO is exempt from s.53 OWRA.

Beds of Navigable Waters Act, R.S.O. 1990, B.4

Section 1 of this Act states:

> *Where land that borders on a navigable body of water or stream, or on which the whole or a part of a navigable body of water or stream is situate, or through which a navigable body of*

water or stream flows, has been or is granted by the Crown, it shall be deemed, in the absence of an express grant of it, that the bed of such body of water was not intended to pass and did not pass to the grantee.

The result of this is that the bed of a navigable body of water is in most cases deemed to be Crown or public land, and the Public Lands Act applies as described above.

Conservation Authorities Act, R.S.O. 1990, C.27

Section 20 of the Conservation Authorities Act states that the objects of an authority are to establish and undertake a program designed to further the conservation, development, restoration and management of natural resources, excluding gas, oil, coal and minerals. The Conservation Authorities Act grants an authority the powers (S. 21) necessary to carry out the program, including the power to erect structures, create reservoirs, control the flow of water, prevent or reduce floods and/or pollution, alter or divert the course of any river or road, and re-align any watermain, gas main, sewer or drain for its purposes.

Under Section 28 an authority may make regulations,

- restricting the use of water in or from lakes, wetlands, rivers and other watercourses;
- prohibiting, regulating or requiring permission of the authority for any interference with existing watercourse channels;
- regulating the location of irrigation ponds;
- regulating construction in any area susceptible to flooding during a regional storm, and defining the regional storm for the purposes of such regulations; and
- prohibiting, regulating or requiring permission of the authority for the placing of fill in any defined area in which, in the opinion of the authority, the control of flooding or pollution or the conservation of land may be affected by the placing of fill.

Although the Act does not bind MTO, MTO staff should communicate with the local conservation authority to ensure that the proposals are acceptable to the authority. Problems may arise concerning the use of the Regulatory Flood for the design of bridges and culverts or highway embankments on flood plains or flood ways.

Section 31(3) of the Act deals with a project of an authority which will interfere with a public road or highway. The authority must file with the Minister of Transportation a plan and description of the project, with a statement of the interference with the highway and how the authority proposes to remedy the interference. MTO will review the project and issue an approval if appropriate. All costs for such a project are borne by the conservation authority unless otherwise agreed.

Section 29(1) (d) empowers an authority to make regulations applicable to lands owned by the authority *"prescribing permits designating privileges in connection with use of the lands or any part thereof and prescribing fees for permits."*

Drainage Act, R.S.O. 1990, D.17

The Drainage Act is a major statute governing the authorization of construction and maintenance of artificial drainage facilities. It is a successor to several other Acts, namely the Municipal Drainage Act, the Ditches and Watercourses Act, the Interprovincial Drainage Act, the Municipal Aid to Drainage Act and the Provincial Aid to Drainage Act.

When a highway requires new or improved drainage, use of the Drainage Act by MTO may offer the following advantages:

- cost of the works is shared among the owners who benefit; and
- maintenance of drains constructed under a by-law passed under the Act is performed by the municipality.

Some of the procedures under the Act relevant to highways are as follows:

- mutual Agreement Drains under Section 2 may be used only when a municipality is the second;
- requisition Drains under Section 3 may be useful in special cases;
- petition Drains under Section 4 of the Act are frequently used;
- the relocation of municipal drains on or adjacent to the highway may be accomplished under Sections 77(2) and 77(3) of the Act; and
- other relocations or improvements to existing drains may be undertaken by the municipality under Section 78.

The following situations illustrate typical uses of the Drainage Act by MTO. The practitioner is referenced to the appropriate Ministry Directive regarding works under the Drainage Act. In situations where MTO is considering improvements to culverts on municipal drains, the municipalities should always be informed, and the desirability of performing the work under the Drainage Act should be assessed on a case by case basis.

It should be noted that the status of a drain may be important when maintenance of the drain becomes necessary. A drain may be considered a private drain if design, construction, or maintenance of the works has not been carried out under any Act or Regulation, such as the Drainage Act, Municipal Act, or Local Improvement Act. In this case the owner has to maintain the drain. On the other hand, if the drain was constructed by by-law under the Drainage Act, the municipality assumes responsibility for maintenance.

Award Drains (Section 3(18))

Award drains were created under the former Ditches and Watercourses Act that was repealed in 1963. They were so named because the work of construction and maintenance was awarded to individual owners along the ditch. Section 3(18) of the Drainage Act provides that an award drain be maintained in accordance with the original award until such drain is brought under the Drainage Act by requisition (Section 3) or by petition (Section 4). Identification of award drains can be made with the help of local residents or the drainage or road superintendent, or by reference to the files of the municipality. With the exception of civil litigation, no mechanism exists to enforce maintenance of an Award Drain.

Mutual Agreement Drains (Section 2)

When two or more owners wish to build or improve a drainage works on their lands, they may enter into a written mutual agreement for the financing, construction and maintenance of a drain under Section 2 of the Act. A legal survey is not required for this type of drain, provided that the land on which the drain is situated is described in the agreement sufficiently for the purposes of registration. A description of the drainage works and its location is also required.

In view of problems encountered by MTO in the enforcement of mutual agreements on subsequent owners of the property involved, MTO's Office of Legal Services recommends that such agreements be entered into only with municipalities.

Mutual agreement drains can be identified at Registry Offices, or by consulting local residents and municipal drainage or road superintendents .

Requisition Drains (Section 3)

Another method of obtaining drainage is by requisition. In this case the owner of land requiring drainage may file a requisition form with the clerk of the local municipality along with a $300 deposit to defray subsequent costs. Requisition drains are subject to the condition that the total estimated cost must not exceed $7,500, exclusive of the cost of crossing lands occupied by the works of public utilities or road authorities. Only lands lying within 750 metres of the drainage works and land lying within 750 metres from the upstream point of commencement of the works may be assessed for costs. Upon filing of the requisition, the council is obliged to appoint an engineer to prepare a preliminary report, which must be accompanied by a benefit cost statement and an environmental statement. Requisition drains are of little benefit to MTO, but may be of use in special circumstances.

Petition Drains

One or more owners of an area requiring drainage by means of a drainage works may initiate a petition for consideration by the council of the municipality. Two major advantages of the use of petition procedures are that costs are shared by those who benefit, and that the responsibility for future maintenance rests with the municipality, although at the expense of the upstream lands assessed for the original construction or improvement of the drain.

MTO may initiate a petition in the following situations :

- where an outlet is required for draining a highway;
- where an improved outlet is required for draining a highway ;
- where relocation of a drain is required for highway purposes, provided that Sections 77(2), 77(3) or 78 are not more appropriate; and
- where a connection to a municipal drain or sewer is necessary for proper drainage of a highway.

Petition Requirements

A petition may be filed with the clerk of the local municipality in which the area requiring drainage is situated by:

(a) *the majority in number of the owners, as shown by the last revised assessment roll of lands in the area, including the owners of any roads in the area;*
(b) *the owner or owners, as shown by the last revised assessment roll, of lands in the area representing at least 60% of hectarage in the area;*
(c) *where a drainage works is required for a road or part thereof, the engineer, road superintendent or person having jurisdiction over such road or part despite subsection 61(5);*
(d) *where a drainage works is required for the drainage of lands used for agricultural purposes, the Director (as appointed by the Minister of Agriculture, Food and Rural Affairs).*

In cases (a) or (b), MTO may or may not sign a petition, depending on the probable benefits or lack thereof. Petitions supported by MTO are to be signed by the appropriate person authorized by MTO pursuant to the Drainage Act and the Public Transportation and Highway Improvement Act.

If the council decides to proceed with a drainage works, it must give written notice to each petitioner and appoint an engineer. (Sections 5(1) and 8(1) respectively).

The Engineers Report An engineer must make an examination of the area requiring rainage and prepare a report for council. This will include, according to Sections 8(1), 13(1), 14 to 38, and 40, the following items:

- definition of the problem, based on the requirements of owners, onsite meetings, review of old reports and personal examination. The problem to be solved is then set forth by the engineer;
- discussion of alternative solutions to the problem, and of the particular scheme recommended;
- plans and profiles of the proposed drainage works, and information on construction, land use, disposal sites, etc.;
- specifications and special provisions governing construction of the proposed drain and associated structures, such as bridges and culverts;
- estimate of costs of materials, labour, plant and equipment for completion of project. The engineer's fee and the administration cost of the contract are included in the total estimate;
- a schedule of assessments prepared for each portion of the drain and for each township separately, where applicable. The schedule should show the following information: the lot and concession number of the assessed parcel of land, the owner's name, the approximate area in hectares, and the cost assessed for benefit and outlet liability. The total of all assessments must be equal to the estimated cost of the project; and
- allowances in the schedule of assessment covering the cost of lands necessary for construction or improvement of the drain, disposal of material, and the site of a pumping station (including access); allowances for crop damage or severance. All allowances to be paid to owners of lands affected.

A review of the drainage engineer's report by MTO staff should take account of the following considerations.

- Compliance with the Drainage Act - in reviewing a drainage report it is worthwhile to check that it complies with the requirements of the Drainage Act, including those listed above.
- MTO Requirements - the work proposed in the report with respect to the highway should satisfy MTO’s requirements. For example a report may specify that a ditch be built alongside an existing two-lane highway. However, if future widening to four lanes is planned, the ditch should be located so as not to interfere with the future construction. Also, where a drain crosses the highway right-of-way, the drainage facility should meet MTO standards and specifications.
- Assessments - assessments against highway property must be commensurate with benefits derived from the drainage works, as discussed in the following Subsections.
- Drainage Assessments - assessments are the means by which the costs of a drainage works are shared among the landowners. Generally assessments are based on benefits received by the landowners as a result of the drainage works; that is, the owner who benefits the most, should pay the most.

The Drainage Act specifies five assessment categories.

- Assessing for Benefit (Sections 1 (Defining Benefit) and 22) - This type of assessment applies where advantages to any lands, roads or buildings as a result of the drainage works

- can be identified, aside from the general improvement of all the lands in the locality. Examples of such benefits are higher market values of property, control of surface or subsurface flow, improved appearance, improved crossings other than those considered as a special benefit, and agricultural improvements such as increased crop production.
- The benefit to roads must be clearly stated in the report. If MTO is assessed for benefit, the responsible Regional office and the District Engineer should conduct a careful review prior to approval of the assessment to ascertain whether the stated benefit is reasonable.
- Special Benefit (Sections 1 (Defining Special Benefit) and 24) - special benefit is defined as *"any additional work or feature included in the construction, repair or improvement of a drainage works that has no effect on the functioning of the drainage works."* Lands may be assessed if special benefits, usually at the request of individual owners, have been included in the drainage works. A footbridge crossing the drain is one example of special benefit. MTO is seldom assessed for special benefit.
- Injuring Liability (Sections 1(Defining Injuring Liability) and 23(2)) - injuring liability is that part of the cost of the drainage works "required to relieve the owners of any land or road from liability for injury caused by water artificially made to flow from such land or road upon any other land or road." (S.1(Defining Injuring Liability)). In other words, if the proposed drain relieves a land owner of a situation where, under common law, he has improperly directed surface runoff on to adjacent lands, then the owner may be assessed for having this situation relieved. Assessment in this case is based on volume and rate of flow of the diverted water. This is a unique type of assessment and is seldom applied.
- Outlet Liability (Sections 1(Defining Outlet Liability) and 23(1)) - outlet liability is *the part of the cost of the construction, improvement or maintenance of a drainage works that is required to provide such outlet or improved outlet*. (S.1 (Defining Outlet Liability)). *Lands and roads that use a drainage works as an outlet, or for which, when the drainage works is constructed or improved, an improved outlet is provided either directly or indirectly through the medium of any other drainage works or of a swale, ravine, creek or watercourse, may be assessed for outlet liability*. (S. 23 (1)). This is normally based on the contributing drainage area, which in the case of roads is multiplied by a weighting factor which may range from 3 or less for a gravel road to 5 for a paved road. Assessments against MTO for outlet liability should be checked to ensure they are acceptable.
- Assessment against Road Authority Section 26 of the Drainage Act states that the road authority shall be assessed for and pay all the increase in cost of the drainage works caused by the existence of the road. For example, if a municipal drain crosses a highway, the road authority would be assessed the cost of providing a new bridge or culvert or modifying an existing structure, as required.

Option To Carry Out Drainage Works on Highways

Section 69 of the Drainage Act provides the road authority with the option to construct drainage works across the highway right-of-way and, where this option is not exercised, allows the municipality to complete the works within the right-of-way in the same manner as other drainage works.

It is still necessary that the municipality obtain consent of MTO (i.e. in the form of an Encroachment Permit) before carrying out any work on provincial highway rights-of-way, as provided in Section 25(1) of the Public Transportation and Highway Improvement Act.

Section 69 states:

(1) *Where a drainage works or a part thereof is to be constructed, improved, maintained or repaired upon, along, adjoining, under or across the lands, permanent way, transmission lines, power lines, wires, conduits or other permanent property of a public utility or road authority, the public utility or road authority may construct, improve, maintain or repair such drainage works or part; and*

(2) *Where the public utility or road authority does not exercise its powers under subsection 1 or does not complete such drainage works or part within a reasonable time and without unnecessary delay, such drainage or part may be completed by the initiating municipality in the same manner as any other drainage works.*

Drain Relocation

Section 77 (2) provides for relocation of a drainage works on or adjacent to the highway right-of-way at the request of the road authority and at the expense of same, upon the report of an engineer appointed by the municipality. The Act requires that this report contain the information listed in Section 8. Section 77(2) allows relocation of a drain, within the highway right-of-way, upon the written opinion of an engineer, appointed by the municipality, that the relocation will have no adverse effects.

Existing Drain Improvement

Section 78 provides that where, for the better use, maintenance or repair of any drainage works, land, or roads, it is considered expedient to change the course of the drainage works or to carry out other specified types of work, the municipality may, without petition but on the report of an engineer, undertake the works. All proceedings with respect to the report are to be the same as on a report for the construction of a drainage works under the Act.

Obstruction Removal

Under Section 80 the municipality has powers concerning the removal of obstructions such as those caused by low bridges, inadequate culverts or washing out of private drains, for which the owner or occupant is responsible. The municipality may also authorize emergency work under

Section 124 where the Minister of Agriculture, Food and Rural Affairs (OMAFRA) declares that such an emergency exists. Section 81 deals with minor obstructions for which the owner or occupant is not responsible.

Drainage Works in Unorganized Territories

Section 123 gives the Minister of Agriculture, Food and Rural Affairs the power to prescribe the manner in which drainage works shall be carried out in territories which do not have municipal organization.

Use of the Drainage Act in Urban Areas

When an attempt is made to use the Drainage Act to satisfy urban drainage needs, such as the provision of storm sewers for a new residential development, MTO, when involved, should bring the matter to the attention of OMAFRA.

Lakes and Rivers Improvement Act, R.S.O. 1990, L.3

The Lakes and Rivers Improvement Act defines the powers and responsibilities of the MNR with respect to regulating the use of and improvements in the waters of the lakes and rivers of Ontario. It is also designed to protect the interests of riparian owners and to manage fish, wildlife and other water-dependent resources.

Section 14(1) states that no person shall construct a dam on any lake or river,

(a) until the location has been approved in writing by the Minister; and
(b) until the plans and specifications thereof have been approved in writing by the (same) Minister.

It is noted in the Act *that "dam" means a dam or other work forwarding, holding back or diverting water*. This is interpreted by MNR to include most bridges, culverts, stream diversions, causeways, embankments, retaining walls, revetments, municipal drains and dikes. The term *"river" includes a river and a stream.*

Although MTO is not bound by the Act, it is MTO’s practice to cooperate with MNR by providing MNR with details of any significant proposals affecting streams in the Province, such as bridges, large culverts, stream diversions or placement of fill.

Section 24 of the present Act may be relevant to the removal of beaver dams which present an unusually severe hazard to a highway.

Section 24 states:

> *Subject to compensation being made as provided by the Minister of Government Services Act for any damage sustained by reason thereof, the Minister may authorize any person employed by or under the Minister, to enter into and upon any land and remove any rocks, stones, gravel, slab or timber jam, dam or part of any dam, rubbish of any kind or other obstruction in any lake or river, the removal of which he or she considers necessary or expedient for the achievement of any of the purposes of this Act.*

Local Improvement Act, R.S.O. 1990, L.26

Works which may be undertaken by a municipality as local improvements include construction, enlargement and extension of sewers, and protection works on banks of rivers and streams or along lakeshores (Section 2 (1)).

Methods of initiating such works are:

- By petition from at least 2/3 of the owners representing at least 1/2 the value of the lots liable to be specially assessed, (Section 11);
- By a vote of 2/3 of all council members, with the approval of the Municipal Board, (Section 8(1)); and
- On recommendation of the Minister of Health or the local Medical Officer, (Section 9). In this case petitions against the works are ineffective and this procedure is not used frequently.

The provisions of this Act are most likely to be applied in urban areas, rural sub-divisions and in the cottage country. The differences in petition procedures from those of the Drainage Act should be noted.

Municipal Act, R.S.O. 1990, M.45

The Municipal Act sets out the powers and responsibilities of municipalities. So far as drainage is concerned, by-laws may be passed by councils for construction, maintenance, alteration, diversion or stopping up of drains, sewers or watercourses. (Section 207 (13)). The same section empowers the municipality to acquire land for any of the above purposes. Obstruction of drains and watercourses may be prohibited by by-law under Section 207(16). The Act also covers several other aspects of municipal drainage.

Planning Act, R.S.O. 1990, c.P.13[7]

Section 3 of the Planning Act authorizes the Minister of Municipal Affairs and Housing to issue policy statements. Pursuant to s.3(5) of the Act, all other ministries are required to "have regard to" such policy statements when exercising authority affecting planning matters. On May 22, 1996, the 6 comprehensive policy statements which had formerly been issued were replaced by a single provincial policy statement. The portions of that policy statement which may relate to drainage are listed below.

s.1.1.1(e) — *A coordinated approach should be achieved when dealing with issues which cross municipal boundaries, including: 1. infrastructure and public service facilities; and 2. ecosystem and watershed related issues.*

s.1.1.3 — *Long term economic prosperity will be supported by: a) making provisions such that infrastructure and public service facilities will be available to accommodate projected growth; and, g) planning so that major facilities (including transportation corridors) are appropriately buffered or separated.*

s.1.3.1 — *Planning for sewage and water systems will recognize that full municipal sewage and water services are the preferred form of servicing for urban areas and rural settlement areas.*

s.1.3.2.1 — *Transportation systems will be provided which are safe, environmentally sensitive and energy efficient.*

s.2.3.1(b) — *Development and site alteration will be permitted in environmentally sensitive areas where it would otherwise be prohibited only if it can be demonstrated that there are no negative impacts.*

s.2.4.1 — *The quality of and quantity of ground water and surface water and the function of sensitive ground water recharge/discharge areas, aquifers and headwaters will be protected or enhanced.*

Public Lands Act, R.S.O. 1990, P.43

Although MTO is not bound by the Public Lands Act, knowledge of the Public Lands Act may be useful to MTO staff involved with municipal stream crossings. Under the Act the Minister of Natural Resources has power to control the erection of any structure on public land; an example would be construction of a pier for a municipal bridge in a navigable stream, the bed of which may be Crown land by virtue of the Beds of Navigable Waters Act discussed below. In such cases MNR may require an application by the municipality for a license of occupation.

[7] This Act has been amended. Refer to the official statute for details.

Tile Drainage Act, R.S.O. 1990, T.8

Although not applicable to MTO, it is useful to be aware of the main objectives of the Act. These are to provide agricultural landowners with the means of obtaining low cost loans for constructing drainage works, and to permit municipalities to borrow money to provide the loans for this purpose.

Other Provincial Legislation Related to Drainage

In specific situations, other provincial statutes and agreements may influence water management issues such as :

Agricultural Tile Drainage Installation Act;
Aggregate Resources Act;
Building Code Act;
Game and Fish Act;
Mining Act;
Parks Assistance Act;
Pesticides Act;
Petroleum Resources Act;
Pits and Quarries Control Act;
Provincial Parks Act;
Pollution Abatement Incentives Act;
Shoreline Protection Act; and
Wilderness Areas Act.

Appendix 2D: Agency Mandates

Table 2D.1 presents a summary of the mandates that arise from legislation administered by the various agencies.

Table 2D.1: A Sample Summary of Agency Mandates Arising from Legislation

Agency	Acts Administered	Functions of Act	Other Agency Involvement
CA	CA Act	• to establish and undertake within a watershed boundary a program designed to further the conservation, restoration development and management of renewable natural resources • primarily concerned with water quality management and erosion • also involved in water related land management	• commenting agency for municipal and county planning • commenting agency under Drainage Act • liaise with MNR, MOE, OMAFRA, Trent Severn Water- way
MMAH	Planning Act	• chief legislative mechanism for governing and providing for municipal land-use planning. • empowers municipality to undertake official plans, zoning by-laws, site plans and subdivision consents	• local municipalities • counties or regions • CA, MNR, MOE, others as commenting agencies.
LMA County/ RMA	Planning Act	• empowered by MMAH to undertake: zoning by-laws, local official plans and amendments • provision for Committee of Adjustment, Land Division Committee	• MMAH, CA, MNR, MOEE, etc. as reviewing agencies
MNR	Public Lands Act	• provides for the regulation, administration, management and use of Crown lands.	
	Beds of Navigable Waters Act	• provides for exemption of certain townships under the Act. • to provide the background by which a water course or waterbody is deemed navigable (public ownership) or not navigable (private ownership) • to protect navigable waters for public use.	
	Lakes and Rivers Improvement Act	• to provide for the use of waters of the lakes and rivers of Ontario and to regulate improvements in them. • provides for public and riparian rights, use, management and perpetuation of fish, wildlife and other natural resources; preservation of natural amenities, ensuring suitability of improvements.	
	Canada Fisheries Act	• conserve and preserve fisheries • regulates the deposit of deleterious substances in water or where the substance will reach the water and negatively affect fisheries. • major thrust is fish habitat protection.	• MOEE where approval required under Ontario Water Resources Act. • other agencies as applicable. • Federal Department of Fisheries and Oceans. • Federal Department of Environment.

Table 2D.1: A Sample Summary of Agency Mandates Arising from Legislation

Agency	Acts Administered	Functions of Act	Other Agency Involvement
MOEE	Environmental Protection Act	• to provide for the protection and conservation of the natural environment. • deals primarily with pollution by contaminants as defined in the regulations. • the betterment of the people of the whole or any part of Ontario by providing for the protection, conservation, and wise management in Ontario of the environment.	• binds the Crown
	Environmental Assessment Act	• review and approve environmental assessments of water and land management undertakings which may have significant effects on the environment. • currently applies to all public agencies unless exempted by regulations.	
	Ontario Water Resources Act	• main legislative instrument for regulating water quality.	
OMAFRA	Drainage Act	• provides for authorization of agreement, petition and requisition drains, and sets out financial arrangements for their construction, maintenance and minor improvements.	• municipalities undertake constructions.
	Tile Drainage Act	• assistance in construction of on-farm tile drainage.	• local municipalities
Trent-Severn Water-way	Navigable Waters Protection Act (only in Trent Severn Waterway - in other parts of Canada there is another designated authority)	• prohibits throwing or depositing any substance in a navigable waterway, including erection and placing of works which may cause impairments to navigation. • pertains to waters under federal jurisdiction.	• administered by TSW for Transport Canada. • applications circulated to Environmental Protection Service, Transport Canada, MNR, MOE

Legend:

CA Conservation Authoirty
MMAH Ministry of Municipal Affairs and Housing
MNR Ministry of Natural Resources
LMA Local Municipality
RMA Regional Municipality
MOEE Ministry of Environment and Energy
OMAFRA Ministry of Agriculture, Food and Rural Affairs

Appendix 2E: Documents Supporting Statutory Mandates

Table 2E.1 presents the support document that arise from legislation administered by the various agencies.

Table 2E.1: Compilation of Policies, Guidelines, and Regulations of Provincial and Municipal Agencies

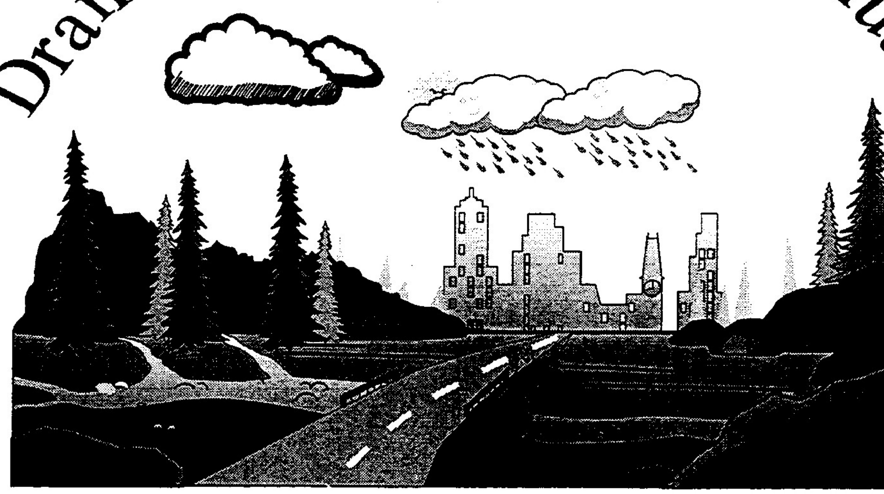
Agency	Policies	Guidelines	Regulations
MOEE	Ontario Water Resources Act Environmental Protection Act Environmental Assessment Act Planning Act "Water Management" (PWQO) Reasonable Use for Groundwater Impact (Policy No. 15-08) Policy on Planning for Sewage and Water Services	Technical Guidelines for Preparing a Pollution Control Plan Guidelines for Preparing EAs Oak Ridges Moraine Planning Guidelines Bay of Quinte RAP Guidelines Guidelines for the Design of Sanitary Sewage Systems Guide for Applying for Approval of Municipal and Private Sewage Works (sections 52,53 of OWRA Chapter 0.40) Manual of Environmental Policies and Guidelines, Vol 1&2 Manual of Policy, Procedures and Guidelines for On-Site Sewage Systems	O. Reg. 358 (R.R.O. 1990) under the EPA (on Sewage Systems) O. Reg. 374/81 under the EPA (on Sewage Systems)
MNR	Planning Act (Natural Heritage Policy, Mineral Resources Policy, and Public Health and Safety Policy) Public Lands Act	Guidelines and Criteria for Approvals under the Lakes and Rivers Improvement Act Fish Habitat Protection Guidelines for Developing Areas (1994) Great Lakes - St. Lawrence Shorelines Technical Guidelines Natural Channel Systems: an Approach to Management and Design (Developmental Draft, 1994) Floodplain Management In Ontario Technical Guidelines	
MMAH	Planning Act Municipal Act	Growth and Settlement Policy Guidelines Guidelines for Preparing Environment Assessments - Land Use Planning Component Making Choices: Guidelines for Alternative Development Standards	
OMAFRA	Drainage Act Agricultural Tile Drainage Installation Act Tile Drainage Act		
Joint MOEE/MNR	Tri-Documents: • Integrating Water Management Objectives with Municipal Planning Documents • Watershed Management on a Watershed Basis • Subwatershed Planning	Interim Stormwater Quality Control Guidelines for New Development Stormwater Best Management Practices Planning and Design Manual	

Table 2E.1: Compilation of Policies, Guidelines, and Regulations of Provincial and Municipal Agencies

Agency	Policies	Guidelines	Regulations
Conservation Authorities (Collectively and Individually)	Conservation Authorities Act Policies in various Watershed Plans ESA/ANSI Policy MTRCA Stream and Valley Corridors Policy	Land Use and Development Policy Guidelines Maitland River Conservation Strategy on Land Use and Development Lake Simcoe Environmental Management Strategy	O. Reg. 404 & 406/83 under the Planning Act O. Reg. 617/86 & 253/89 on Fill, Construction and Alteration of Waterway
Joint MNR/MOEE/ MMAH/MTO/ ACAO/MEA/ UDI		Guidelines on Erosion and Sediment Control for Urban Construction Sites Urban Drainage Design Guidelines	
RTAC		Drainage Manual (highway drainage design practises)	
Local/Regional Municipalities	Policies in various • Official Plans • Secondary Plans • By-laws	Subdivision Design and Servicing Standards	
		Guidelines for SWM in municipalities, e.g.: • Hanlon Creek • Laurel Creek • Joshua Creek	
DFO	The Federal Policy for the Management of Fish Habitat (1986) Canada-Ontario Memorandum of Intent on the Management of Fish Habitat (1989)	Habitat Conservation and Protection Guidelines (1993)	

Ministry of Transportation
Ontario

MTO Drainage Management Manual



Chapter 3
Developing and Evaluating Design Alternatives

Drainage and Hydrology Section
Transportation Engineering Branch
Quality and Standards Division

3 Table of Contents

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Purpose of This Chapter

The purpose of this chapter is to present the methodology for developing drainage designs for highway projects and illustrate how the development of drainage designs "fit" within the Highway Planning and Design Process. Figure 3.1 presents the stages of development of drainage designs as part of the highway planning and design process.

The focus, in this chapter, will be on outlining the thought process for the preparation of drainage designs. The design considerations, levels of detail, and the choice of numerical methods required for the analysis associated with the different stages of design will be discussed. Design details of specific drainage components will be covered in Part 2 of the manual.

Due to the interdisciplinary nature associated with the development of a drainage design, it is intended that this chapter clearly identify areas where consultation with, and involvement of, other professional disciplines is required. Information that is not directly related to drainage is included only for:

- information purposes;
- to familiarize the drainage practitioner with the language of other disciplines; and
- to familiarize the drainage practitioner with the issues that are shared between the different disciplines.

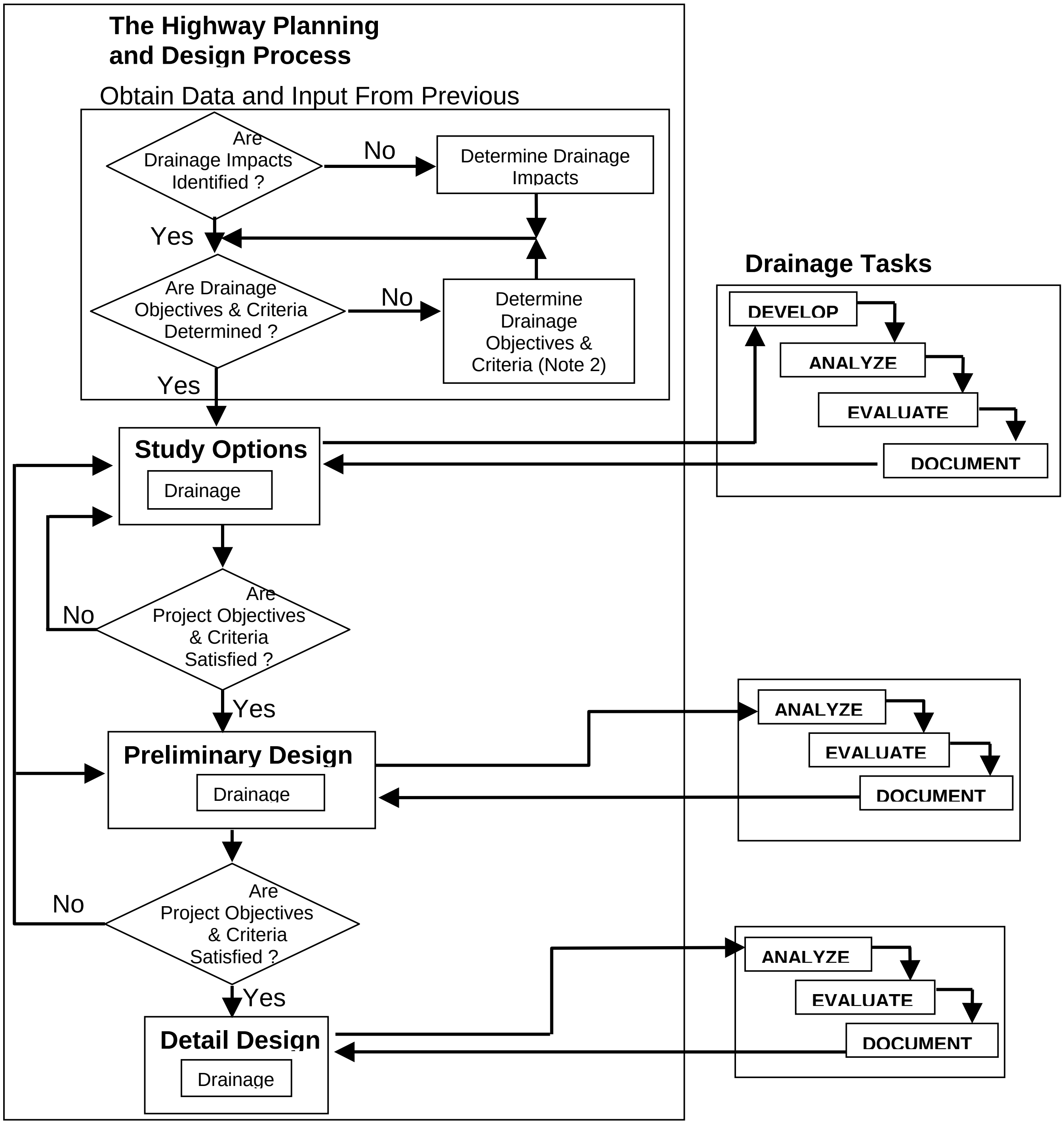
Solutions to impacts not directly related to drainage design are outside the scope of this manual, and are discussed in other MTO documents such as the:

- *Environmental Manual, Fisheries*; or
- *Environmental Manual, Sediment and Erosion Control (Working Draft)*.

When designing solutions to drainage related impacts, advantages gained through the interdisciplinary team approach cannot be overstressed.

It should be noted that specific design objectives, criteria, and options for an individual highway project, including the drainage management components, will be established by the project through the class environmental assessment process. The material presented in this chapter is intended to be read and used in this context.

Figure 3.1: The Drainage Design Process



Introduction

The development of highways will, in most cases, impact the surrounding environment. One of the means of addressing these impacts in the design of highways is the adoption of modern drainage management techniques. The concept of modern drainage management is based on the belief that the most effective means of addressing the impacts of a highway development is through the adoption of a design methodology that will :

- achieve the objectives of the highway development; and,
- account for the limitations and constraints of the surrounding natural, social and economic environment.

The design methodology can be accomplished in three steps, as follows.

1. Identification of drainage impacts.
2. Determination of objectives and criteria for drainage design.
3. Development of the drainage design.

Chapter 2 provides guidance on how to complete the first two steps. This chapter takes the next step. It describes the methodology that can be used to incorporate the output from the previous two steps, into the design of a preferred drainage option. Any drainage option should, therefore, satisfy the constraints and requirements of the highway alternative being considered, as well as the constraints and limitations set by the surrounding natural and social environment.

As described in Chapter 2, the drainage-related objectives and criteria for a highway project are derived based on the potential watershed impacts of the highway project and associated drainage works. These objectives and criteria are also a reflection of the governing laws, codes, policies, standards, and guidelines. Therefore, a wide range of issues may be involved in the design of drainage works. This will require the involvement of an interdisciplinary team which may include engineers, planners, biologists and landscape architects.

In order for a drainage design to satisfy the wide range of objectives and criteria established for the project, design criteria will need to be considered for the following:

- hydrology;
- hydraulics (including geomorphology);
- soil erosion;
- water quality
- terrestrial biota;
- aquatic biota; and

- socioeconomic factors.

This chapter will focus on the design methodology related to hydrology, hydraulics, and water quality. Guidance on design considerations related to aquatic biota, terrestrial biota and socioeconomic factors are beyond the scope of this manual and will be left to other relevant MTO manuals and external references to provide the required guidance. It is important to note, however, that the consideration of aquatic biota, terrestrial biota and socioeconomic factors is an integral part of developing the project objectives and criteria. These criteria provide the principles that guide the design of the highway drainage works.

Introducing Drainage Design within the Highway Planning and Design Process

As was previously discussed in Chapter 1, drainage design is part of the highway planning and design process. Decisions made regarding the drainage design are not made in isolation. As illustrated in Figure 3.1, the design process begins once the impacts of the proposed highway project have been identified, and the objectives and criteria for drainage design have been established.

The design may be performed in three stages. The main intent of the division into the three stages is to allow the thought process and level of effort, to proceed from a broad and preliminary level to a narrow and more detailed level. These three stages can be accomplished in one single design assignment, or more than one assignment, as circumstances require.

- **Selection of study options** is the first step in the design process. A number of options may be feasible. Each option is then analyzed, evaluated and the results documented. The main purpose for this procedure is to identify possible options and eliminate any options that do not satisfy the objectives and criteria. Additional information may be identified to assist in further analysis at later stages of development. At the end of this stage, options that merit further investigation through preliminary design should be identified and the results documented.
- **Preliminary design** is a more detailed investigation of the study options identified. At this stage, a more detailed level of analysis and evaluation is needed to determine the most suitable option(s) that satisfy the design objectives and criteria prior to proceeding to detail design. At the end of this stage documentation of the preferred design(s) (preliminary design report) may be prepared.
- **Detail design** is the design analysis and evaluation of the preliminary design(s), performed to select the preferred option and document the design details. The level of this analysis and evaluation is much more detailed, and the preferred option selected should satisfies the project objectives and criteria.

Regardless of which design stage is being considered, there are four tasks to be done. These tasks are:

- developing the drainage design;
- analysing the drainage design;
- evaluating the drainage design; and
- documenting the drainage design.

Developing the drainage design is introduced below. Since this task is the main scope of this manual, a detailed discussion that specifically focuses on developing the drainage design for the preliminary and detail design stages, is presented in subsequent sections (refer to Figure 3.3). Analysis of the drainage design will be discussed throughout the manual. The other tasks, evaluation and documentation, are part of the broader highway planning and design process, and will not be discussed in any great detail within this manual.

The level to which each of the tasks is completed will depend on the scope and scale of the project. Figure 3.1 illustrates the linkage between the three design stages and the four main tasks.

Developing the Drainage Design

A highway project will generally include one or more of the following elements:

- horizontal and vertical alignments;
- widenings;
- interchanges;
- ancillary facilities, such as car pool areas; and
- rehabilitation.

Depending on the scale of the highway project, each of the highway elements may include other components, of which drainage is one component. Drainage components can be grouped as follows.

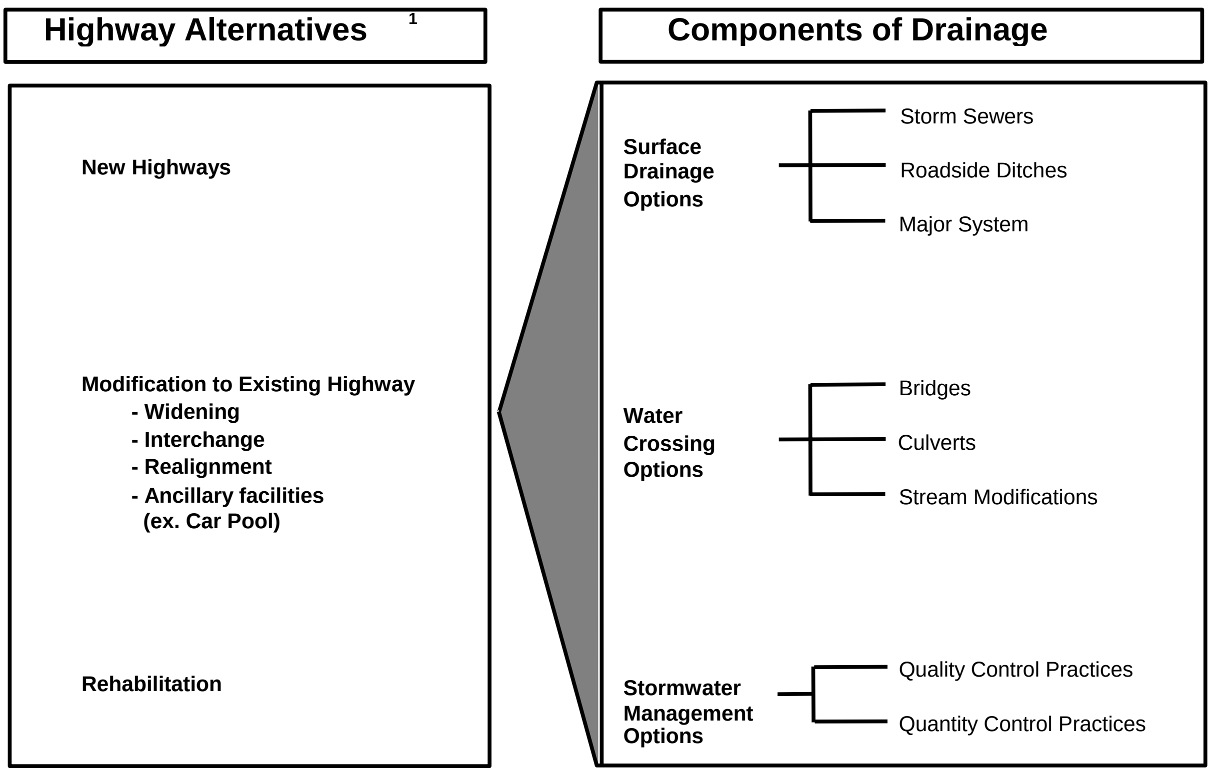
- **Water crossing components** are drainage works that are associated with a highway crossing of a stream, river, creek or lake . These components include culverts, bridges, and stream modifications, such as, diversions, channelization, and enclosures.
- **Surface drainage components** are drainage works that collect and transport stormwater runoff from the highway right-of-way and surrounding catchment, to a receiving body of water such as stream, river, or lake. These components include ditches, storm sewers, and the major flow system.
- **Stormwater management components** are drainage works that are needed to control stormwater runoff. These components are either quantity control or quality control facilities. Quantity control facilities detain runoff for a required period of time before releasing it at a specified rate. Water quality control facilities treat surface runoff and reduce the amount of pollutants released to the environment. In some cases either quantity or quality control is needed. In other instances, both are required and can be provided in separate or combined facilities.

To develop a drainage option, identify the different drainage components required for the highway project. The selection of different combinations of components will result in different drainage options. Figure 3.2 illustrates the drainage options associated with different highway elements.

In developing the drainage options it is important to keep in mind that at the end of the process, all of the components must fit together logically and effectively. This includes consideration of possible cumulative and associative effects. In doing so, the highway elements must be considered in conjunction with the drainage options, to ensure conflicts do not exist. An overview of the entire system can often be overlooked as one focuses on the development of individual components.

A detailed discussion on developing the drainage design is provided in subsequent sections of this chapter (refer to Figure 3.3).

Figure 3.2: Highway Elements and Associated Components of Drainage Options



Notes: **1** Highway alternatives are developed as outlined in the *Regional Planning and Design Management Manual* (MTO, 1993) which follows the class environmental assessment procedure (see Figure 1.1).

Analyzing the Drainage Design

The analysis of the drainage design is not separate from the overall analysis of the highway project. The analysis may include the determination of the following:

- size, number, configuration, type of material, and location of each drainage component of

the related highway element;
- estimate of natural, social and economic impacts of the drainage option;
- land requirements;
- construction cost;
- road closings or detours; and
- long term maintenance requirements.

The analysis of drainage designs takes place throughout the study option, preliminary design and detailed design stages. When analyzing drainage study options, the level of detail required in the calculation is limited. The goal is to identify the possible drainage components and determine the approximate values of quantifiable design parameters. At this stage the practitioner may be able to evaluate and eliminate any options that are not feasible, or that clearly do not meet the design objectives and criteria. On the other hand, at the preliminary design stage the design of drainage facilities usually requires accurate identification of design parameters such as flow rate, storage capacities and water elevations, as well as the comparison of post development conditions to existing hydraulic and hydrologic conditions. Depending on the nature of the project, considerable effort could be spent in such an analysis. This analysis would provide the data and information required for the evaluation and selection of a preferred study option(s), prior to proceeding to detail design. Analysis at the detail design stage may be similar to the analysis performed at the preliminary design stage. However, the analysis will be more detailed and will give complete consideration to detailed site conditions. In some cases, a more accurate design method or computer model may have to be used which requires significantly more effort.

Evaluating the Drainage Design

The evaluation of the drainage design is part of the overall evaluation of the highway project and will, therefore, not be discussed in any detail. This task helps to identify the best option or alternative, through a comparison of their relative values. The best option is one that:

- achieves the same results as other options but at a lower cost; or
- costs the same but has fewer adverse effects; or
- costs more but has additional benefits that justify the extra cost.

For a detailed discussion on the process of evaluation of drainage designs, refer to Appendix 3C.

Documenting the Drainage Design

For each highway alternative that includes a drainage option, a design report may be prepared at each of the design stages. These reports will serve to document the results of the development, analysis and evaluation tasks at each stage. In general this report should include all data and calculations relevant to the design(s) of the preferred drainage option(s), including input and output data and information, computer input and output files, drawings of the drainage plan(s) in relation to the highway development plan(s) and the contributing watershed, all necessary dimensions, water levels, protective works, and other information related to the design.

The items identified above are by no means complete. Examples of reporting requirements can be found in MTO's *Guidelines for Preparation of Hydraulics Reports*. More details are also provided in Chapters 4 and 5 in Part 2 and Chapter 8 in Part 3.

Concluding Notes

When evaluating the process presented in Figure 3.1 and Figure 3.2, the following points need to be considered.

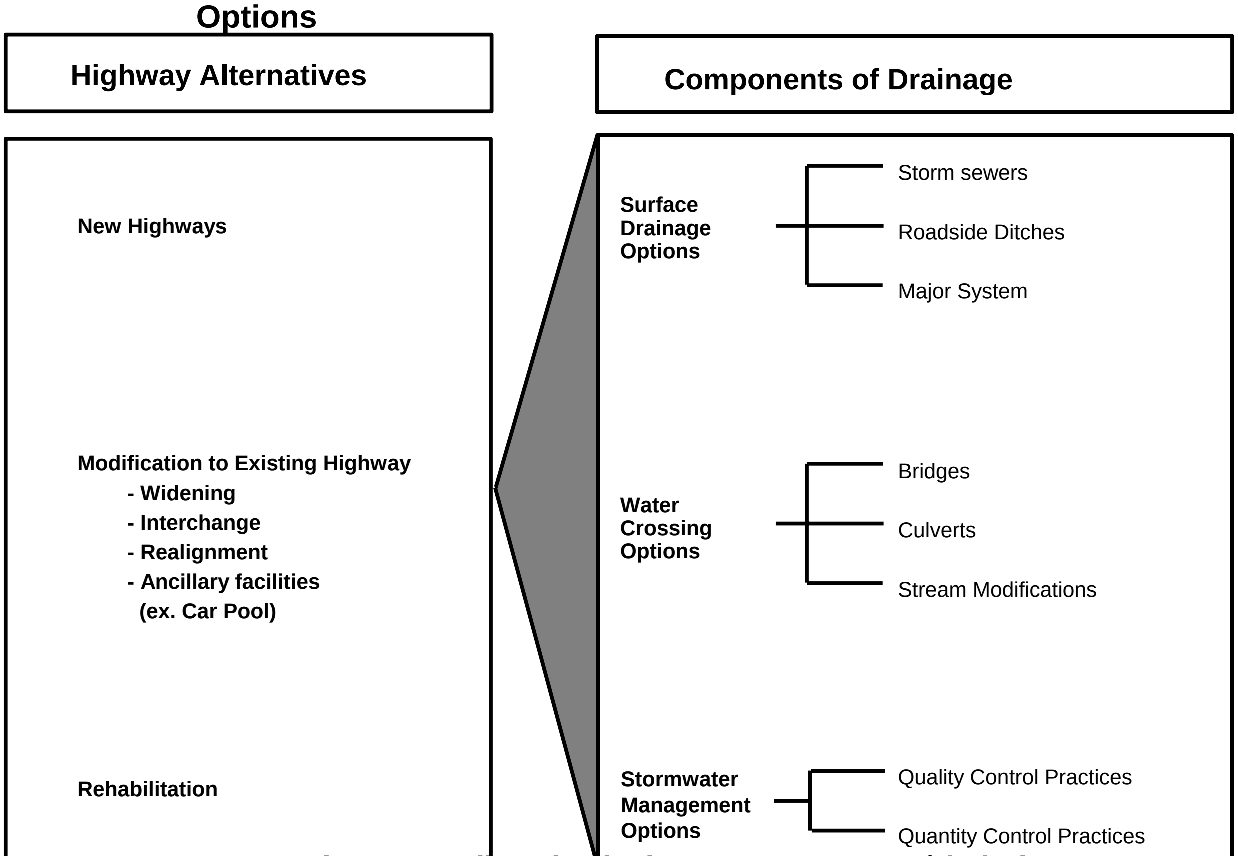
- **The process may be iterative.** Design requirements, public concerns, scientific information, natural environment issues and awareness of environmental processes may change. Consequently, objectives and criteria may have to be modified and new design options considered which can result in the design procedure being an iterative process.
- **The process is flexible.** All steps in the highway planning and design process are not always needed nor followed in the specific order of presentation (i.e. as in Figure 3.1). The process is not rigid and need not be divided into separate stages. For instance, the completion of the study options and preliminary design may be done in one step if, for example, only one design option is being considered. The exact sequence of the process will depend on the scale and nature of each project.
- **The process includes drainage.** Drainage design is an integral part of the highway planning and design process. Correspondingly, the primary purpose of the *Drainage Management Manual* is to provide highway design practitioners with guidance on the design of drainage works in support of the highway planning and design process.
- **For rehabilitation, drainage works may be the project.** In most instances, the highway project will involve the design or construction of highway elements, such as widenings, realignments, interchanges, and the associated drainage component. However, in some cases the project may only involve drainage works. For example, a project may involve the analysis of culvert crossings to determine effectiveness, potential liabilities and long term maintenance requirements. Considerations, in such a case, will mostly be drainage related.

It is outside the scope of this manual to discuss specific design procedures related to the broader highway planning and design process, and, more specifically, how each of the four tasks are completed within the three design stages. It will be the responsibility of the practitioner to apply the drainage design procedures and guidance appropriately.

A Quick Reference for Developing a Drainage Design

In developing a highway design, considerations may be given to a number of alternatives, each including one, or combinations of the highway elements discussed in the proceeding section. Drainage design will be a component of the design associated with each alternative. When developing the drainage design for each of the highway alternatives, one or more drainage options may be possible, with each alternative and option consisting of various drainage components. Figure 3.2, which has been repeated here for convenience, illustrates the highway alternatives and associated components of drainage options.

Figure 3.2: Highway Alternatives and Associated Components of Drainage Options



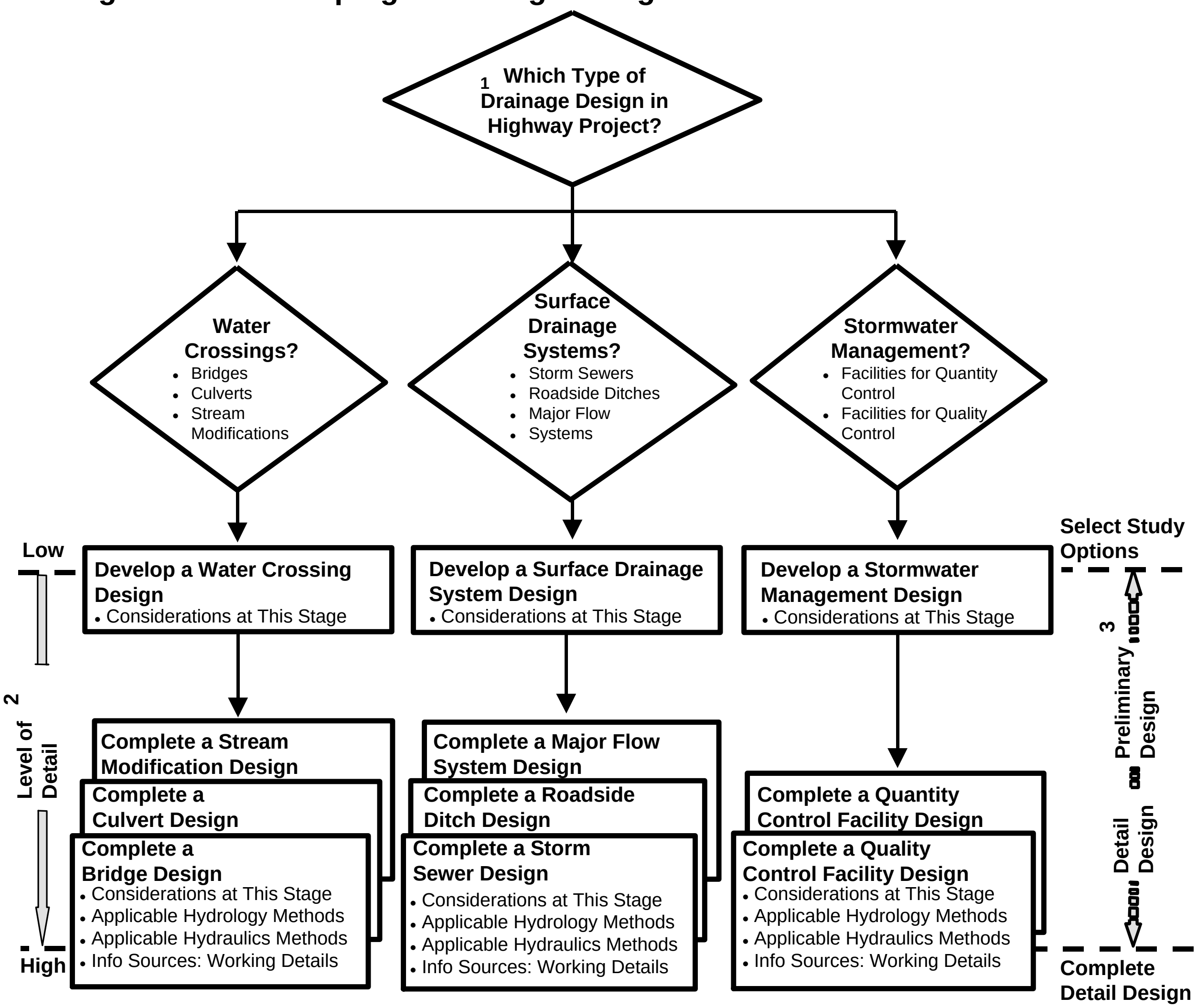
Note: In most instances the project will involve the design or construction of the highway alternatives; however, in some cases the project may be drainage specific.

The development of the drainage design consists of two fundamental parts:

- review considerations for developing a drainage option; and
- complete the design of the drainage component.

The subsequent sections present the various considerations that can be applied to these steps. The organization is presented on Figure 3.3. All the various considerations are summarized in Table 3.1a to Table 3.1e.

Figure 3.3: Developing a Drainage Design



Notes:
1. The highway planning and design process is flexible. Study options, preliminary design and detail design can be completed in three separate stages, or less.
2. If the preliminary design stage is applicable, the level of detail will depend on the scope, scale and nature of the highway project.
3. The chapters for detail design (i.e. in Part 2) present design methodologies and considerations.

Table 3.1 a: Bridge Crossing Design

<table>
<tr><th colspan="2">Considerations for Developing a Bridge Crossing Design</th></tr>
<tr><td>The Need for Stream Channel Modifications
Adverse Impacts Associated with Stream Channel Modifications
• Stream Channel Stability
• Watershed hydrology
• Stream channel hydraulics
• Riparian land owner rights
• Aquatic habitat
• Terrestrial habitat
• Water quality (sediment and water temperature)
Location of the Water Crossing
• Stream channel stability and self-adjustment
• Crossing at aggrading/degrading stream channels
• Crossing at alluvial fans and deltas
• Crossing at braided channels
• Crossing at a stream confluence
• Local stream channel modifications
• Crossing at wetlands and lakes
• Possible problems with floating debris
• Factors related to the water crossing and the highway
• External constraints
• Navigation requirements
• Water crossings on sag curves</td><td>Water Crossings and the Highway Profile
• Freeboard
• Relief flow
• Relief flow and the highway profile
• Minor access under water crossings
Permanent Erosion Control Measures
Structure Performance
Soil and Foundation Considerations
Long Term Maintenance Considerations
Factors to Review when Considering a Bridge Crossing Option
• Location and alignment
• Pier and abutment location and alignment
• Dual Parallel Bridges
Relative Advantages of Bridges as Compared to Culverts
Relative Advantages of Culverts as Compared to Bridges
Locating Fish Habitat Structures within a Stream
Consider Data Needs
• Proposed crossing
• Existing structures
• Local information
• Soil information
• Fish migration data</td></tr>
<tr><th colspan="2">Completing a Bridge Crossing Design (also refer to Chapter 5)*</th></tr>
<tr><td>Considerations at this Stage of the Design
• Hydraulic problems at bridge crossings
• Span arrangement
• Piers details
• Abutments
• Superstructures
• Soffit elevation
Applicable Hydrologic Methods
• Stream flow rates
• Backwater assessment
• Hydrologic models</td><td>Applicable Hydraulic Methods
• Sizing of waterway opening
• Backwater assessment
• Hydraulic jump
• Vertical drops, scour protection, flow control
Information Sources and Working Details
• Flow conveyance and backwater;
• River ice;
• Scour;
• Fish passage;
• Debris flow;
• Energy dissipation;
• Erodible channels;
• Stream channel lining material; and
• The hydraulic design of fish habitat structures.</td></tr>
</table>

* Note: The detail design chapters do not only present design methodologies, they also present consideration in detail design.

Table 3.1 b: Culvert Crossing Design

Considerations for Developing a Culvert Crossing Design

The Need for Stream Channel Modifications
Adverse Impacts Associated with Stream Channel Modifications
- Stream Channel Stability
- Watershed hydrology
- Stream channel hydraulics
- Riparian land owner rights
- Aquatic habitat
- Terrestrial habitat
- Water quality (sediment and water temperature)

Location of the Water Crossing
- Stream channel stability and self-adjustment
- Crossing at aggrading/degrading stream channels
- Crossing at alluvial fans and deltas
- Crossing at braided channels
- Crossing at a stream confluence
- Local stream channel modifications
- Crossing at wetlands and lakes
- Possible problems with floating debris
- Factors related to the water crossing and the highway
- External constraints
- Navigation requirements
- Water crossings on sag curves

Water Crossings and the Highway Profile
- Freeboard
- Relief flow

Water Crossings and the Highway Profile (con't)
- Relief flow and the highway profile
- Minor access under water crossings

Permanent Erosion Control Measures
Structure Performance
Soil and Foundation Considerations
Long Term Maintenance Considerations
Factors to Review When Considering a Culvert Crossing Option
- Location and alignment
- Culvert profile
- Culvert embedment
- Culvert length

Relative Advantages of Bridges as Compared to Culverts
Relative Advantages of Culverts as Compared to Bridges
Locating Fish Habitat Structures within a Stream
Consider Data Needs
- Proposed crossing
- Existing structures
- Local information
- Soil information
- Fish migration data

Completing a Culvert Crossing Design (also refer to Chapter 5)*

Considerations at this Stage
- Common hydraulic problems
- Open footing versus closed invert culverts
- Culvert Material
- Culvert shape
- Multi-barrel culverts
- Culvert profile
- Culvert length
- Culvert Safety Concerns
- Fish passage design flow
- Clay seals
- Other-tailwater level, improved inlets, end treatments and embankment fills

Applicable Hydrologic Methods
- Stream flow
- Peak flow computation
- Hydrograph computation / Hydrologic models
- Flood plain impacts

Applicable Hydraulic Methods
- Sizing of waterway opening
- Inlet and outlet control
- Hydraulic jump
- Backwater assessment
- Vertical drops, scour protection, flow control

Information Sources and Working Details
- Flow through culverts
- River ice
- Scour
- Fish passage
- Debris flow
- Energy dissipation
- Erodible stream channels
- Stream channel lining material
- The hydraulic design of fish habitat structures

* Note: The detail design chapters do not only present design methodologies, they also present consideration in detail design.

Table 3.1 c: Stream Modification Design

Considerations for Developing a Stream Modification	
The Need for Stream Channel Modifications **Adverse Impacts Associated with Stream Channel Modifications** • Stream Channel Stability • Watershed hydrology • Stream channel hydraulics • Riparian land owner rights • Aquatic habitat • Terrestrial habitat • Water quality (sediment and water temperature) **Location of the Water Crossing** • Stream channel stability and self-adjustment • Crossing at aggrading/degrading stream channels • Crossing at alluvial fans and deltas • Crossing at braided channels • Crossing at a stream confluence • Local stream channel modifications • Crossing at wetlands and lakes • Possible problems with floating debris • Factors related to the water crossing and the highway • External constraints • Navigation requirements • Water crossings on sag curves	**Water Crossings and the Highway Profile** • Freeboard • Relief flow • Relief flow and the highway profile • Minor access under water crossings **Permanent Erosion Control Measures** **Structure Performance** **Soil and Foundation Considerations** **Long Term Maintenance Considerations** **Locating Fish Habitat Structures within a Stream** **Consider Data Needs** • Proposed crossing • Existing structures • Local information • Soil information • Fish migration data
Completing a Stream Channel Modification Design (also refer to Chapter 5)*	
Considerations at this Stage • Natural character of the stream and the flood plain • Natural flow patterns • Erosion control measures • Fish habitat provisions • Channel bed and bank stability **Applicable Hydrologic Methods** • Stream flow • Peak flow computation • Hydrograph computation • Flood plain impacts • Stream channel routing • Hydrologic models	**Applicable Hydraulic Methods** • Sizing and flow capacity • Hydraulic jump • Backwater assessment • Vertical drops, scour protection, flow control **Information Sources and Working Details** • Remedial erosion control works • Suitable stream channel sections • Type of lining material, vegetative or otherwise • Stream channel bends, meanders and alignments • Energy dissipators • Scour protection • The hydraulic design of fish habitat structures • Fish passage

* Note: The detail design chapters do not only present design methodologies, they also present consideration in detail design.

Table 3.1 d: Surface Drainage Design

Considerations for Developing a Surface Drainage Design	
Selection of the Minor Drainage System **Advantages of Storm Sewers Compared to Roadside Ditches** **Advantages of Roadside Ditches Compared to Storm Sewers**	**Checking the Major Drainage System** **Checking the Receiving System** **Consider Data Needs**
Completing a Storm Sewer Design (also refer to Chapter 4)*	
Considerations at This Stage • Conveyance capacity • Catchbasin spacing capacity • Surcharging • Impact to receiving stream channel system **Applicable Hydrologic Methods** • Peak flow computation • Separation of major and minor system hydrographs • Hydrologic models for receiver impacts assessment	**Applicable Hydraulic Methods** • Sizing and flow capacity • Surcharge and backwater analysis • Water surface profiles in the receiver **Information Sources and Working Details** • Sewer material • Sewer elevations • Hydraulic considerations (size, slope, velocity) • Sewer inlet and outlet conditions • Exfiltration to local subgrade • Safety for people and vehicles • Sewer accessories
Completing a Roadside Ditch Design (also refer to Chapter 4)*	
Considerations at This Stage • Conveyance capacity • Backwater potential • Erosion control (drop structure, channel lining) • Impact to receiving stream channel system **Applicable Hydrologic Methods** • Peak flow computations • Hydrologic models for receiver impacts assessment **Applicable Hydraulic Methods** • Sizing and flow capacity • Backwater analysis • Water surface profiles in the receiver	**Information Sources and Working Details** • Ditch cross-section and lining • Profile, invert and crest elevations • Very flat terrain • Very steep terrain • Roadside safety • Porous soil • Limited availability of right-of-way • Entrances to adjacent property • Aesthetic considerations
Completing a Major Drainage System Design (also refer to Chapter 4)*	
Considerations at This Stage • Major storm conveyance and depth of flow • Flooding at highway sags • Overland flow routes to receiver • Erosion **Applicable Hydrologic Methods** • Peak flow computation • Hydrograph computation • Major flow routing • Backwater analysis • Hydrologic models	**Applicable Hydraulic Methods** • Flow capacity and depth • Backwater assessment **Information Sources and Working Details**

* Note: The detail design chapters do not only present design methodologies, they also present consideration in detail design.

Table 3.1 e: Stormwater Management System Design

Considerations for Developing a Stormwater Management Design	
The Current Approach to Stormwater Management **Stormwater Impacts Associated with Highways** • Stormwater quality impacts • Stormwater quantity impacts **Typical Contaminants Associated with Highway Stormwater** **Identification of Best Management Practices Suitable for Highways** • General considerations • BMPs suitable for highway development	**Receiving Water Based Quality Control Criteria** **Assessing the Need for Stormwater Quality and Quantity Control** • The need for stormwater quality control • The need for stormwater quantity control **Location and Layout of the Stormwater Management System**
Completing a Stormwater Quality Control Facility Design (also refer to Chapter 4)*	
Considerations at this Stage • Size • Length to width ratio • Detention time • Inlet and outlet configuration • Emergency bypass location, type and capacity • Maintenance access • Special safety and maintenance requirements • Grading and planting strategy • Other (freeboard, side slope, embankment stability, bottom grade)	**Applicable Hydrologic Methods** • Single event simulation • Derived probability distribution method • Continuous simulation **Applicable Hydraulic Methods** • Reservoir sizing • Reservoir routing **Information Sources and Working Details** • Wet and extended detention ponds
Completing a Stormwater Quantity Control Facility Design (also refer to Chapter 4)*	
Considerations at this Stage • Size • Detention time • Inlet and outlet configuration • Emergency bypass location, type and capacity • Maintenance access • Special safety and maintenance requirements • Grading and planting strategy • Other (freeboard, side slope, embankment stability, bottom grade)	**Applicable Hydrologic Methods** • Single event modelling **Applicable Hydraulic Methods** • Reservoir sizing • Reservoir routing **Information Sources and Working Details** • Dry detention ponds

* Note: The detail design chapters do not only present design methodologies, they also present consideration in detail design.

Developing a Water Crossing Design

The considerations that are presented in this section for developing a water crossing design, are best suited for the preliminary stages of the highway planning and design process. If the highway project is at the later stages of design, refer to the following sections.

- Completing a Bridge Crossing Design;
- Completing a Culvert Crossing Design; or
- Completing a Stream Channel Modification Design.

Often, the highway water crossing will involve some stream channel modification with a culvert or bridge placed at the actual highway water crossing location. For instance, the approach or exit section of a bridge may require realignment to improve flow direction; or stream channels are diverted and combined to limit the number of actual water crossings, reducing the number of bridges and culverts that are needed. In some cases, a stream channel modification can be utilized to avoid the need for a highway water crossing. In general, the development of a water crossing design involves two basic tasks.

- **Develop the stream modification** if a stream channel modification is preferred. If a stream channel modification is preferred, determine the form of modification that should be used (e.g. stream realignment, diversion or enclosure).
- **Develop the water crossing.** Assess the best location for the crossing, and determine the form that the crossing should take (i.e. bridge or culvert).

In completing these tasks, the practitioner needs to fit the water crossing options to the highway alternative. This can be achieved by:

- reviewing the various considerations for developing the water crossing design;
- considering the possible water crossing options; and
- reviewing the possible considerations related to the highway.

A summary of the tasks is presented in Table 3.2.

Table 3.2: Fitting the Water Crossing Options to the Highway Alternative

Task	Considerations for Developing the Water Crossing Design	Possible Water Crossing Options	Possible Highway Considerations
Develop the Stream Modification	• the need for stream channel modifications • adverse impacts associated with stream channel modifications	• stream diversion • stream enclosure • stream realignment	• highway alignment • interchange location
Develop the Water Crossing	• location of the water crossing • general considerations for water crossings • bridge crossings characteristics • culvert crossings characteristics • relative advantages of bridges, as compared to culverts • relative advantages of culverts, as compared to bridges	• bridge • culvert	• highway alignment • highway profile • interchange location

The latitude with which the practitioner has in completing these tasks will also depend on two other factors.

- **The highway project** can be grouped as new highways, modifications to an existing highway, or a rehabilitation (refer to Figure 3.2). For new highways, the practitioner may have flexibility in selecting water crossing options, as different routes may be under consideration. Using the project objectives and criteria as a guide, the number of water crossings, the form of the crossing and the location of the crossing may all be adjusted to suit the different alternatives that may exist. In contrast, the practitioner may not have the same flexibility for a modification to an existing highway. For instance, in the case of widenings, the number and location of the crossings may be set, and perhaps the form of the crossing is also set. In such a case, there is very little that a practitioner can do with respect to developing the design; so, the project will likely be focussed on design specifics.
- **The planning and design process** may include various stages (refer to Figure 3.1). The flexibility that a practitioner has in developing a water crossing design will be greatly influenced by the stage in which the planning and design process has progressed to, when the water crossing is first considered. To properly develop water crossing designs, it is critical to consider the water crossing as early on in the planning and design process as possible, preferably at the study options or the preliminary design stages rather than at the later design stage.

It is also important to recognize that the planning and design process is iterative. As the project evolves through the various stages and steps, the water crossing design should be evaluated to ensure compliance with the objectives and criteria. A re-adjustment of the water crossing design should occur whenever it has been determined that the objectives or criteria have not been fully complied with.

The following subsections present the various considerations to be applied in developing the water crossing design:

- the need for stream channel modifications;
- adverse impacts associated with stream channel modifications;
- location of the water crossing;
- water crossings and the highway profile;
- permanent erosion control measures;
- structure performance;
- soil and foundation considerations;
- long term maintenance considerations;
- factors to review when considering a bridge crossing;
- factors to review when considering a culvert crossing;
- relative advantages of bridges, as compared to culverts;
- relative advantages of culverts, as compared to bridges;
- locating fish habitat structures within a stream; and
- consider data needs.

The Need for Stream Channel Modifications

A thorough search for solutions that could avoid stream channel modifications is an important task and needs to be completed at the earliest possible stage of design. A thorough search should involve all affected professional disciplines, and internal and external offices and interests. If a stream modification is essential, it may take one of these forms:

- several stream channels are diverted and combined to form a single stream channel;
- a stream channel is realigned to avoid intersection with the highway;
- a stream channel is modified to accommodate a bridge or culvert;
- an entire stream channel is replaced with a culvert or sewer; or
- modifications are required to mitigate an on-going erosion problem.

Further variations of the above forms include lined stream channels, transitions (expansions, contractions, weirs and confluences), and energy dissipators (drop structures, stilling basins and baffled chutes).

Guidance on the cases where stream channel modifications may be justified, provided there is no reasonable alternative such as relocating the highway, are as follows.

- To reduce an excessive angle of skew (> 45°), which is both costly and structurally undesirable.
- To eliminate or reduce excessive encroachment of fill on the stream channel. This may occur with high fills at skew crossings or where the channel is alongside the highway.

- To eliminate a serious erosion problem at the highway or downstream property.
- On rare occasions, to reduce the high water elevation at or upstream from a water crossing.
- To reduce the number or length of water crossings at a complex interchange.
- To reduce the number of crossings. For example, to eliminate the need for two crossings where a tributary stream crosses the highway to enter the main stream a short distance downstream.
- To reduce the height of fill by moving the crossing away from a high valley side.
- To reduce the number of crossings of a meandering stream flowing generally parallel to the highway.

Adverse Impacts Associated with Stream Channel Modifications

Stream channel modifications may impact the following (for more detail, refer to Table 2.1 and Table 2.2 in Chapter 2):

- stream channel stability;
- watershed hydrology;
- stream channel hydraulics;
- riparian land owner rights;
- aquatic habitat;
- terrestrial habitat; and
- water quality.

Stream Channel Stability

Modifications to stream channels may cause an impact to the overall stability of the stream channel. Impacts may include changes to shape, width, depth, length, sinuosity (i.e. stream channel meandering), and sediment loads. The impacts are the result of changes in the watershed geology, topography, soils, vegetation and precipitation. To minimize impacts, stream channel stability must be assessed before recommending any stream channel modification scheme . Considerable judgement must be used when designing, locating and sizing stable natural stream channels. Chapter 9, recommends ten steps for assessing stream channels. If stream channel stability is not assessed, some possible consequences are:

- bed degradation and bank slumping upstream;
- bed aggradation and increased channel shifts downstream;
- destruction of favourable fish and wildlife habitat such as pools and shoals;
- increased erosion of the remaining channel(s);
- adverse effects on upstream and downstream channel conditions causing high channel maintenance costs.

Watershed Hydrology

Any stream channel modification scheme may impact the hydrology of the watershed or catchment area. To assess any potential impacts to watershed hydrology, a hydrologic assessment should be completed for the existing case, as well as for all of the recommended schemes. The hydrologic assessment should include the following.

- Ensure that all tributary areas are included and their boundaries are correctly determined.
- Determine major and minor flow paths, and flow patterns which affect the design flow calculations.
- Determine the affect modification has on hydrograph timing, peak flow and shape.
- Calculate design flows of all design frequencies including high flows, low flows and fish passage flows, where applicable. It is increasingly important to consider low flows for environmental protection purposes such as erosion control, stormwater quality management, base flow management and wildlife habitat requirements. Refer to Appendix 3A for a discussion on hydrologic computational procedures.

Stream Channel Hydraulics

Any stream channel modification scheme may impact the hydraulics of the stream channel. To assess any potential impacts to stream channel hydraulics, a hydraulic assessment should be completed for the existing case, as well as for all of the recommended schemes. The hydraulic assessment should include the following:

- a stream channel flow analysis (velocities, water surface profiles, subcritical/supercritical flow);
- possible effects of the drainage works on the velocity frequency distribution for a flow event (e.g. 2yr stream flow hydrograph);
- effects on stream channel routing (stage versus storage and stage versus discharge);
- effects on stream channel stability (tractive forces and scour potential);
- the possibility of more severe ice jamming due to elimination of stream bends;
- effects on the severity of ice jamming or flooding due to the elimination of relief flow; and
- an assessment of regulatory flood levels for the existing case, and determine if any potential impacts to the regulatory flood levels may be caused by the stream channel modification schemes.

Riparian Land Owner Rights

The rights and uses of riparian land owners must be assessed when any water crossing or stream channel modification scheme is being considered. Generally, a riparian land owner has the right to have water flow in its natural state with regard to both quantity and quality, and may put the water from the stream channel to any reasonable use. Any stream channel modification that may increase or decrease the quantity or quality of water that traverses a riparian land owner's property, could prove to be an infringement upon their rights. Riparian land owner rights are generally dictated by common law precedents. Refer to Chapter 2, Appendix 2B for a detailed discussion on riparian land owner rights.

Aquatic Habitat

Some potential impediments to aquatic habitat, fish passage and spawning areas that may result from improperly designed stream channel modification schemes, are as follows:

- stream channel flow velocities are increased and flow depths are reduced when steeper stream channel slopes and smoother lining materials are used;
- natural fish resting areas are eliminated;
- prolonged or short-term flooding is increased;
- large artificial stream channel bed drops are created;
- water temperature increases when vegetation adjacent to a watercourse is removed;
- damage to downstream habitat caused by sedimentation of spawning and hatching grounds;
- restrictions on fish passage where check dams or channel drops are used (alternatively, a special provision, such as a fish ladder, could be included, although this would be costly);
- restrictions on fish passage if erosion protection (e.g. riprap) is not sufficiently imbedded into the stream bed; and
- increases in water temperature caused by the use of impervious linings (i.e. natural groundwater is reduced due to subsequent reductions in infiltration).

For details, refer to the MTO *Environmental Manual, Fisheries*

Terrestrial Habitats

Stream channel modification schemes may affect:

- trees, shrubs, grasses and legumes;
- vegetation (lichens, mosses, etc.);
- crops;
- wildlife (mammals, reptiles, amphibians, etc.);
- insects; and
- birds (migratory and resident).

Water Quality

Generally, a stream channel modification scheme may impact water quality by increasing water temperatures or by increasing sediment loading.

Location of the Water Crossing

Considerations for the location of water crossings include:

- stream channel stability and self-adjustment;
- crossings at aggrading and degrading channel;
- crossings at alluvial fans and deltas;
- crossings at braided channels;
- crossings at a stream confluence;
- local stream channel modifications;
- crossing at wetlands and lakes;
- possible problems with floating debris;
- factors related to the water crossing and the highway;
- external constraints;
- navigation requirements; and
- bridges on sag curves.

Stream Channel Stability and Self-adjustment

One of the most important hydraulic factors governing the location of a water crossing is the stability of the stream channel. Chapter 9 provides a detailed discussion on stream channel stability. The following effects may influence the location and design of a water crossing.

- If an erodible channel is constricted by a crossing, the bed will tend to scour.
- If a channel is steepened by elimination of meanders etc., the channel will tend to return to the flatter slope by degrading its bed.
- When bank protection is used to prevent channel shifts at one location, erosion may be accelerated elsewhere.
- Erosion at one location in a channel is usually accompanied by deposition at another.
- Over a period of many years, a channel may move back and forth several times over the same area.
- An increase of dominant discharge (e.g. due to diversion of flow from another basin) will increase the channel depth and width.

Crossings at Aggrading and Degrading Channels

Aggrading and degrading channel reaches are generally unstable and unpredictable and should be avoided wherever possible. Aggradation and degradation are discussed in Chapter 9.

Crossings at Alluvial Fans and Deltas

Fans and deltas are discussed in more detail in Chapter 9. Crossings of fans should be avoided whenever possible. If a fan crossing is unavoidable, the same solution as for a braided channel (below) may be feasible. However, frequent excavation of the channel may be required to keep the water crossing waterway open. Delta crossings should be carefully located with the aid of air photos, site inspection and local information.

Crossings at Braided Channels

Crossings of braided channels should be avoided if possible. Works to combine the sub-channels into a single channel should be considered if relocation is not possible.

Crossings at a Stream Confluence

The stability of stream channels located near confluences is sometimes uncertain because material deposited by floods may cause channel shifts in either stream. Furthermore, the direction and distribution of the flow may vary with the discharge and stage of the stream, necessitating careful study to determine the best compromise for the structure location and highway alignment. The history of channel changes at such locations should be investigated.

Local Stream Channel Modifications

Local stream channel modification may be required to accommodate the water crossing. Local modifications include the following:

- straightening one stream channel bank or portions of the stream channel bed;
- straightening both stream channel banks and bed;
- widening the stream channel banks;
- removing natural meanders;
- dyking;
- widening the stream channel banks; and
- deepening, flattening or steepening the stream channel bed.

Local modifications are usually composed of a number of drainage elements including lined stream channels, transitions (expansions, contractions, weirs and confluences) and energy dissipators (drop structures, stilling basins and baffled chutes). The previous section, Adverse Impacts Associated with Stream Channel Modifications, should be reviewed as the considerations listed within that section can also apply to local stream channel modifications. Consideration should also be given to natural channel techniques.

Crossing at Wetlands and Lakes

Crossings of provincially significant natural heritage features, lakes and tidal waters should be avoided if possible; however, if unavoidable, culverts, bridges and causeways should be designed to maintain the natural flow pattern. The use of equalizer culverts should be considered, and the creation of stagnant areas should be avoided.

For additional guidance refer to the MTO *Environmental Manual, Fisheries*, and to the Provincial Policy Statements for Natural Heritage and Natural Hazards.

Possible Problems with Floating Debris

- When selecting a crossing location, try to avoid a split channel or channel bars, as these locations often capture floating debris.
- Where floating debris may be severe, consideration of an alternate crossing location may be advisable. If avoidance of the site is not practical, the design should consider measures to facilitate the passage of debris (e.g. the waterway opening configuration and size).
- Smaller culverts may create problems under some circumstances.
- A bridge may be preferred over a culvert. A single span culvert may be suitable but it likely will require further study to confirm its suitability. A multi-cell culvert should be avoided.
- For some cases, debris deflectors may be recommended to control debris flow.

Factors Related to the Water Crossing and the Highway

- Foundation type and depth.
- Limitations on grade imposed by adjacent properties, topography etc.
- Construction cost.
- Maintenance costs.
- Class of highway; traffic volume, seasonal use only etc.
- Safety of travelling public. Availability of relief flow in extreme floods.

External Constraints

- Requirements of other users of the stream, including hydroelectric facilities and water takings.
- Existing and proposed flood control works.
- Pipeline, sewer lines and other services.

Navigation Requirements

Navigation requirements often constrain the height and width of a water crossing waterway opening. Bridge pier locations and alignments should account for navigation requirements.

Water Crossings on Sag Curves

Water crossings should not be located at the bottom of sag curves, when possible, because of the high damage hazard in the event of an extreme flood. The possibility of surface ponding also exists.

Water Crossings and the Highway Profile

Since it is impractical to design a water crossing to convey the maximum probable flood, a certain amount of risk in terms of structural failure, property damage and loss of life always exists.
It is beyond the scope of this manual to present methods specific to risk and economic analysis. However, in general, it should be realized that a risk or economic analysis is a tool that can be used in making decisions as to the optimum water crossing design. The final decision should be based on the judgement of a team of professionals, and is often influenced by intangible factors to which a dollar value cannot be assigned. General considerations that apply to the water crossing and the highway are as follows:

- freeboard;
- relief flow;
- relied flow and the highway profile; and
- minor access under water crossings.

Freeboard

In establishing the optimum highway profile, it is necessary to balance engineering and constructions costs against the need to keep the highway passable during flood events. The need to protect the water crossing and upstream property during floods must also be considered. This balance may be achieved by establishing the highway grade at a specified height (i.e. freeboard) above the high water elevation; or, by placing the highway grade at an elevation based on a predetermined frequency of overtopping. Guidance in establishing freeboard is provided below.

- Standard freeboard in Ontario (see OHBDC, 1991) shall be not less than 1.0 m for freeways, arterials and collectors, and 0.3 m for other highways such as township highways.
- Freeboard is measured from the edge of through traffic lanes to the design high water level (refer to OHBDC, 1991).

- The 1.0 m freeboard for freeways, arterials and collector highways includes an allowance for a moderate amount of backwater. If the backwater is unusually large, the freeboard should be measured from the upstream high water elevation (including backwater).
- Freeboard for pedestrian or bicycle access paths (refer to OHBDC, 1991) passing through multi-use water crossings, shall not be less than: 1.0 m from the normal water level for spans of more that 6.0 m; and, 0.5 m from the normal water level for spans of 6.0 m or less.

Relief Flow

Relief flow is flow that bypasses the main waterway opening by passing over the approach highway or through one or more relief structures. Generally, an embankment operates as a broad-crested weir when overtopped, and has a very large potential overflow capacity. This provides a "safety valve" against bridge or culvert failure in the event of an extreme flood. It also provides a means of reducing backwater during ice jams or extreme floods, and in some cases can permit a considerable reduction in costs. The principal disadvantage of relief overflow is the inconvenience and possible hazard to highway users. The most economical water crossing, where non-hydraulic conditions permit, is one at which: the normal design flood passes safely through the waterway opening without flooding the approach highway; and, an extreme flood (greatly exceeding the normal design flood) flows over the highway. This "relieves" the bridge or culvert and produces the following benefits and disadvantages.

- Advantages of relief flow are listed below.

    - As soon as flow over the highway commences, the flow velocity through the waterway opening remains steady, and the possibility of scour failure is reduced.
    - Flow over the highway minimizes backwater and its effects on upstream properties. This may be particularly beneficial for the Hazel or Timmins Regulatory floods (i.e. the waterway opening may be designed for the normal design flood while the overall crossing, with relief flow, satisfies the Regulatory flood requirements).
    - A washed-out highway (resulting from an extreme flood) may be brought back into service much more quickly and economically than a washed-out bridge or culvert.
    - The size of flood that can accommodate relief flow over the highway, rather than through a relief structure, is unlimited.
    - The availability of relief overflow can greatly reduce the upstream effects of a blockage to the main channel by ice or debris.
    - If a bridge or culvert structure becomes inundated, relief overflow can reduce much of the lateral pressure and reduce the risk of failure.

- Disadvantages of relief flow are listed below.

    - The principal disadvantages of flow over the highway are the inconvenience and possible risk to the highway user. Although the inconvenience of infrequent flooding of highways is generally accepted by the public, particularly on minor highways, traffic hazards on main highways during overflow periods should be minimized whenever possible by warning signs or highway closures. This may be difficult and sometimes impossible on small streams experiencing flash floods.

- Frequent overflow resulting from an excessively low approach grade may necessitate recurring repairs to the shoulders, fill and possibly the highway. These problems may be overcome by providing an appropriate freeboard, providing erosion protection, or, on existing highways, by raising the grade by a carefully selected amount. Infrequent repairs to the highway are usually acceptable and are quicker and less costly than repairs to washed out structures.

Relief Flow and the Highway Profile

The elevation of the approach grade should be established by adding the specified freeboard to the design high water level. For hydraulic purposes, the optimum highway profile at a water crossing slopes down from the edge of a valley, across the stream and then levels off across the flood plain. This allows plenty of relief flow area, but also keeps the crossing clear of the high water elevation, and avoids the crest curve that would occur if the crossing were near the centre of the flood plain. Additional guidance on relief flow and the highway profile is provided below.

- Make the relief flow section as long as practically possible.
- Avoid locating a highway sag opposite a nearby downstream building that might be damaged by concentrated overflow or by ice passing over the highway.
- Locate the relief flow section well away from the water crossing(s) (e.g. 3 to 4 times the water crossing height) so that the structure(s) will not be damaged in the event of a highway washout.
- Relief culverts may be advisable if the main water crossing is subject to blockage by ice or debris; or, if required for local drainage of a wide flood plain. Experience has shown that relief culverts are of limited value, except in rare circumstances.
- Where a large flow along the highway embankment is anticipated, the approach grade profile should parallel the flow line along the upstream face of the embankment. This applies mainly to a crossing skewed to the flood plain, and to a crossing at the apex of a meander.

Minor Access Routes under Water Crossings

In addition to conveying flow, waterway openings may function as passages for pedestrians, motorized vehicles, wildlife or livestock provided that the waterway is appropriately designed. In these cases, consider the guidance provided below.

- Ministry procedures should be followed when determining whether a multipurpose water crossing is acceptable and how the costs should be shared.

- The cost savings should be weighed against the possible disadvantages of multiple use, such as the potential hazards to people passing through the water crossing. Safety experts and local authorities should be consulted.
- Provision should be made for easy access to the water crossing.
- Installations should be vandal proof, and lighting should be provided where necessary.
- The freeboard of the pathway should be agreed upon by all concerned parties.
- If the access route profile is placed too low, it may be liable to frequent erosion damage and other maintenance problems (e.g. mud, ice, debris). On the other hand, if the access route profile is unnecessarily high, it may significantly reduce the waterway opening.

Permanent Erosion Control Measures

Permanent remedial erosion measures are used to reduce or remedy existing stream channel erosion problems. Erosion problems include:

- bank erosion (e.g. slumping, sloughing, undermining or undercutting) ; or
- stream channel bed erosion (scour).

Remedial measures include vegetation cover (i.e. soil bioengineering), lining treatments, retaining walls, bank drainage diversion, buffer strips, energy dissipators, drop structures, and culvert outlet and inlet treatments.

The use of harder measures (e.g. drop structures, energy dissipators) should not be selected without considering softer measures (e.g. soil bio-engineering, natural channel techniques) first.

Structure Performance

- It is good practice to monitor structure performance during or after large flows. Performance analysis may reveal under or over-design, and may help to reveal widespread design practice problems (e.g. sometimes a culvert washout may have been caused by an extreme flow, well above the design flow, even though the culvert may have been adequately designed).
- Washouts may also result from lesser flows where there are design deficiencies, such as improperly designed spread footings or scour protection.
- Timely maintenance may reduce the need for costly repairs or replacement. Examples of problems to be identified early and corrected are undermining, distortion, inlet lifting, and obstruction by debris and erosion.

Soil and Foundation Considerations

Soil and foundation considerations are the realm of soil and geotechnical engineers. A qualified geotechnical engineer should be consulted. Some soil and foundation considerations are presented to raise general awareness of soil engineering in water crossing design.

- The potential for piping failure.
- Evidence of groundwater problems.
- Possible scour problems.
- Uplift pressures.
- Side slope stability.

Long Term Maintenance Considerations

- Input from maintenance operators, especially when any new design approach or technology is proposed (e.g. maintenance staff to form part of the design team).
- Consider stream channel gradients: slopes that are flat could result in ponding and excessive plant growth which could lead to reduced capacities and potential flooding. Also, very steep slopes could result in stream channel erosion and deposition, reducing stream channel capacities.
- Consider stream channel side slopes: steep side slopes could result in slumping which could block or obstruct flow.
- The provision for an access road (approximately 5 m wide) along the top of the stream channel for maintenance vehicles.
- Written instructions for maintenance should be prepared as part of the design for non-standard drainage facilities. Instructions should include the original design objectives, expected performance of the facilities, and the frequency and extent of maintenance and inspection procedures.
- Items in inspection reports that may be of interest to the designer generally include stream channel bed and bank erosion, and siltation; lining conditions; presence and condition of undergrowth; river aggregation/degradation; abnormal settlement; cracking or spalling of concrete; condition of concrete joints; abnormal foundation leakage; foundation undermining; and description of slide, sloughing and sudden subsistence.

Factors to Review When Considering a Bridge Crossing Option

The considerations presented within this section are of a preliminary nature, and are more suited to any of the following scenarios:

- drainage options are under development for a water crossing location;
- the form of the water crossing (i.e. bridge or culvert), has not yet been determined;

- preliminary considerations are required to determine if a bridge crossing is a viable option; or
- considerations are required to develop a preliminary bridge layout and configuration.

If more detail is required, refer to the section, Completing a Bridge Crossing.

Location and Alignment

- If possible, locate the crossing on a stable reach of stream channel.
- A relatively straight reach is preferred, but a suitable crossing may sometimes be found at the apex of a stable bend impinging on the valley side.
- Care should be taken that the cost of relocating the proposed highway to a stable reach does not outweigh the cost of providing training works on an unstable channel section.
- Avoid obvious problem areas such as areas prone to landslip, which may usually be identified from air photos.
- On highly meandering channels, locate the highway such that the upstream channel loop is far enough away that it will not fold against the highway in the foreseeable future. Relocation of the highway or channel should be considered if necessary to avoid future remedial costs that may be much higher than the cost of relocation.
- Align the structure opening with potential high-velocity flows from an upstream structure or dam spillway. Conversely, align the structure such that the outlet flow will not adversely affect downstream highways, railways or other property. These remarks apply also to relief structures.
- Consider the width of fill at skewed crossings to avoid encroachment on the channel. Take possible future widening of the highway into account.
- If subsoil materials are highly variable (e.g. bedrock on one side of the stream and deep muskeg on the other), select the optimum location based on the consideration of both foundation and hydraulic requirements.

Pier and Abutment Location and Alignment

- Construction of piers on spread footings in erodible channels introduces the possibility of a future scour failure. Therefore, subject to structural and cost considerations, the use of piers on spread footings in such channels, should be minimized or avoided altogether. Piers should also be avoided in very steep, fast-flowing streams transporting gravel or boulders.
- In bridge extensions, avoid placing piers opposite the upstream openings or constructing a single-span extension upstream from multiple spans.
- Piers and abutments should normally be aligned with the main channel. If the flow alignment differs considerably between low and high stages, or the future alignment is uncertain due to probable channel shifts, circular pier shafts should be considered.
- Training works may be needed if channel shifting is a serious problem.

- Abutments on a straight stable channel, are normally placed equidistant from the stream banks.
- A closed abutment on the outside of a stream bend should be placed at or back from the toe of bank, which should be protected against erosion. Any fill slope should be aligned with the toe of the stream bank.
- Piers should be aligned to all flows if possible. Circular pier shafts may be used where the direction of flow varies considerably from low to high stages. Skews should normally not exceed 45° or preferably less.
- If the channel is curved, the effect of the structure outlet velocity on the outer downstream bank should be considered when selecting the skew angle for the piers.
- Debris accumulations preventing fish passage, should be avoided by proper spacing and placement of bridge piers, and by providing vertical clearance above the design flood.
- Full consideration should be given to the potentially harmful effects of stream diversions, which should be constructed only where a reasonable alternative is not feasible (refer to the section, Adverse Impacts Associated With Stream Channel Modifications).

Dual Parallel Bridges

- The backwater produced by a parallel pair of identical bridges is larger than that for a single bridge; but, not as much as would result if the bridges were considered separately.
- If the bridges are more than one bridge opening width apart, backwater should be calculated as if they were single bridges.

Factors to Review When Considering a Culvert Crossing Option

The considerations presented within this section are of a preliminary nature, and are more suited to any of the following scenarios:

- drainage options are under development for a water crossing location;
- the form of the water crossing (i.e. bridge or culvert), has not yet been determined;
- preliminary considerations are required to determine if a culvert crossing is a viable option; or
- considerations are required to develop a preliminary culvert layout and configuration.

If more detail is required, refer to the section, Completing a Culvert Crossing.

Location and Alignment

- Some of the considerations for bridge location and alignment may apply to culvert crossings.
- Significant stream channel modifications that are needed to accommodate a culvert, should

be avoided, wherever possible.

- A straight culvert located on, and aligned with, the natural stream is desirable to avoid uneven scour of open footing culverts and uneven silting of multi-barrel culverts, and to reduce head losses if velocities are high.
- If it is necessary to break the culvert alignment in order to conform with the natural stream alignment, the culvert should be curved in plan or should have angular bends not exceeding 15 degrees at intervals of 15 m. In such cases the possibility of debris blockage should be considered.
- To avoid silting of the barrel on the inside of the bend, it is desirable that mutli-barrel culverts be straight and aligned with the upstream segment of the stream.
- The selection of culvert location and alignment at difficult sites is best done in the field with the aid of air photos and survey plans, and should take account of the width of fill and of any future widening.
- Where flow from a roadside ditch (e.g. on a steep grade) is to be discharged into the culvert, the culvert entrance geometry should be checked to ensure that the flow will not over-shoot the culvert entrance.

Culvert Profile

- Slopes of culverts on uniform grade are usually made parallel to the natural slope unless the gradient is almost flat, in which case the culvert grade may be level.
- Slopes can be modified to improve culvert performance or reduce velocities. In such cases, care should be taken to avoid: aggradation or degradation of the upstream segment of the stream; erosion of the downstream fill or channel; or, loss of aquatic habitat.

Culvert Embedment

- Common practice is to place the floor of a closed invert culvert slightly below the natural stream bed in order to:

    - permit future deepening of the stream;
    - reduce the possibility of undermining; or
    - provide for the passage of fish.

- In deciding on the invert, or depth of embedment, it is important to consider its effect on the future stability of the stream, upstream or downstream of the crossing.
- Embedding the culvert can create or perpetuate upstream degradation. In such cases a drop structure may be required. “Deep” embedments should be artificially filled with gravel.
- Experience has shown that deposits in properly embedded culverts are normally cleaned out by moderate floods.
- On aggrading streams, the culvert invert should be placed no lower that the stream bed.

Additional vertical clearance should be considered to allow for future build-up of the bed material.

Culvert Length

- The length of a culvert should be sufficient to keep the fill from obstructing the waterway. Culvert barrels may be shorter when wingwalls or headwalls are used to retain fill.
- To avoid uplift pressures or fish passage problems, the length should be minimized. If extra length is required for future widening, uplift pressures should be checked and appropriately addressed.
- If a culvert does protrude from an embankment, consideration should also be given to the potential for bank erosion, and safety with regards to errant vehicles.

Relative Advantages of Bridges, As Compared to Culverts

- Bridges may be more economical than culverts with high fills.
- Bridges present less disruption to fish, wildlife, wetlands and aquatic environment.
- Bridges permit easy access along valley park systems, thereby eliminating a traffic hazard for park users at highway crossings (i.e. multi-use crossings).
- Backwater at bridges during major floods is normally less.
- Bridges are usually less susceptible to blockage by debris or ice.
- Bridges may be designed to present less obstruction to navigation.
- Bridges generally cause less disruption to the environment during construction.
- Widening of the bridge (parallel to stream) does not significantly increase backwater.
- The extensive lengthening of culverts to accommodate future highway widenings, may increase backwater.
- The bridge channel invert may be lowered in the future with no great problem, subject to scour protection and foundation design (may not be viable for spread footing piers). The future lowering of the channel invert is limited at closed invert culverts and further deepening could be costly.
- The highway surface, at culverts under shallow fills, may require frequent maintenance because of differential frost heave and settlement.
- Open footing culverts are often susceptible to scour failure.
- Improperly designed or constructed steel culverts are prone to a variety of major problems, including uplift and distortion.
- Improperly designed projecting culvert ends can present a serious hazard to vehicles running off the highway.
- Some culvert types are susceptible to abrasion and corrosion, reducing their service life.
- Rigid culverts are susceptible to separation at the joints, possibly leading to undermining and failure.
- Larger head differentials across culvert embankments are conducive to failure caused by

excessive seepage and piping through fill.

Relative Advantages of Culverts, As Compared to Bridges

- The highway surface at culverts requires less maintenance (except possibly under very shallow fills). Maintenance of the bridge deck surfaces is often costly and difficult.
- Closed invert culverts generally require a smaller waterway opening.
- For properly designed culverts, the risk of scour failure of a closed invert culvert is negligible.
- The highway surface at culverts is not subject to the local icing often experienced on bridge decks, which can create a traffic hazard.
- The highway grade at a culvert can be raised in future. The future raising of the highway grade may be difficult or impossible for some types of bridge structures (i.e. the bridge may have to be replaced).
- Culverts require less structural maintenance than bridges.
- Hydraulic capacity of culverts can sometimes be increased by adding an improved inlet.
- Differential settlement between the approach fill and the structure often causes a bump at the ends of the bridge deck.
- The bridge superstructure is susceptible to damage or failure due to buoyancy, drag and impact forces during extreme floods.
- Bridge deck drainage systems often create significant maintenance problems.
- Spill through bridge fill slopes are susceptible to erosion damage.

Locating Fish Habitat Structures within a Stream

In parallel with the hydraulic design of a water crossing, the fish biologist and other allied professionals will investigate the biological conditions and other conditions of the site, and use their expertise and applicable reference materials to establish the need for, and the design of, fish habitat structures (refer to the MTO *Environmental Manual, Fisheries*). The relative suitability of any potential location selected to house a fish habitat structure should be examined as part of the hydraulic design of the water crossing and will dependent on the factors presented below.

- The geomorphic characteristics of the stream, and the relative stability, or instability, of the stream can be estimated using the ten step procedure that is presented in Chapter 9.
- Stream erosion (i.e. scour), and sediment transport and deposition, may be detrimental to the fish habitat structures. Conversely, the structures may alter the erosion and sedimentation processes of the stream. Methods presented in Chapter 9 can be used to investigate if erosion or sedimentation will be a problem within the stream reach.
- Ensure that hydraulic processes within the stream will not impact the fish habitat structure. Preferably, habitat structures would be located on a straight stretch of the stream (a river bend is not considered to be a desirable location). Habitat structures should be designed to

withstand the design flow velocities and provide little obstruction to flow.
- If habitat structures are proposed to be near a bridge or culvert, potential adverse impacts on the structural integrity and hydraulic performance of the bridge or culvert should be investigated.
- Structures are placed at irregular intervals and in low flow areas.
- Consider construction limitations and post construction monitoring.

Consider Data Needs

For completeness, a brief discussion on data needs associated with water crossings is presented within the following section. Additional data needs for detail design are presented in Chapter 5.

Proposed Crossing

Information on the proposed crossing should be based on a field inspection and on interviews with local residents, municipal officials, maintenance personnel, government agencies, and any other available sources. The following information should be considered:

- details of properties or structures which may be affected by the proposed crossing (e.g. main floor elevation, elevations of well tops upstream of highways, etc.);
- past debris or ice jamming events;
- existing or potential icing problems;
- existing bank erosion and deposition;
- stream bed degradation;
- past or possible future channel alterations;
- stream bed material and rock outcrops;
- past flood elevations (with information on abnormal events);
- controls such as lakes, waterfalls, dams or larger streams;
- alternative locations for new crossings;
- description of channel and flood plain roughness, required to estimate roughness coefficient(s) (including sketch and/or photograph); and
- photographs of existing erosion, properties close to the flood level, and any other special features (may be valuable both for design purposes and in the event of future claims).

Existing Structures

In the design of a new structure, it is often beneficial to consider the hydraulic performance history of existing water crossings. The age of an existing structure and its performance over time is one of the best means of determining its adequacy (e.g. for a fifty year old structure, there is a 64% probability that it has experienced a flow exceeding the 50 year flow). Any information should be based on inspections and on interviews with residents, maintenance staff and other sources. The following site information should be recorded where relevant:

- location and type of structure;
- estimated age;
- dimensions of waterway (normal to flow);
- high water levels upstream and downstream;
- ice or debris problems;
- relief flow over highway;
- any remediation of past problems;
- downstream controls affecting flow and tailwater levels at the water crossing;
- bed material;
- hydraulic damage or repairs to the water crossing;
- excessive erosion;
- past washouts of structure or highway;
- relevant history of structure, such as past channel deepening, degradation or dam washouts;
- the capacity of the water crossing;
- other relevant details including special inlets, outlets and erosion control structures; and
- photographs of relevant features.

Local Information

The importance of interviewing local MTO maintenance staff, local residents and municipal officials, conservation authorities and MNR staff, and other knowledgeable people, cannot be over stressed; since, they will, in many cases, be able to supply information unobtainable in any other way. High water levels in particular should be ascertained or checked from these sources.

Soils Information

The designer should know the nature of the subsoil material underlying the stream bed unless it is obvious that it is sound bedrock, or other material which will create no problem. Organic material should be given special attention, and a sound knowledge of the scourability of the soil is essential. Detailed foundation investigations should be carried out for bridges and large culverts, unless it is certain they will be founded on sound bedrock. A geotechnical specialist will be able to provide guidance for specific data needs.

Fish Migration Data

A fish biologist should be consulted for specific data needs. Also, refer to the MTO *Environmental Manual, Fisheries)*. Useful fish migration data are:

- species of migrating fish, size and swimming speed;
- locations of spawning beds, rearing habitat, and food-producing areas upstream and downstream from the site;
- description of fish habitat at the proposed crossing;
- dates of start, peak and end of migration;
- average flow depth and width during migration; and
- average date of maximum annual flood.

Completing a Bridge Crossing Design

The design of a bridge structure must integrate structural and foundation considerations with the hydraulic considerations of the bridge waterway opening. This section will present the hydraulic considerations for bridge crossings, under the following headings:

- considerations at this stage of the bridge crossing design;
- applicable hydrologic methods for bridge crossing design;
- applicable hydraulic methods for bridge crossing design; and
- information sources and working details for bridge crossing design.

In conjunction with this section, review the section Factors to Review When Considering a Bridge Crossing Option.

Considerations at this Stage of the Bridge Crossing Design

Design considerations for bridge crossings include the following:

- hydraulic problems at bridge crossings;
- span arrangement;
- pier details;
- abutments;
- superstructures; and
- soffit elevation.

Hydraulic Problems At Bridge Crossings

Significant problems that may be experienced at bridge crossings are as follows.

- Bridge failures or structural damage caused by undermining of the foundation by scour, degradation or dredging.
- Partial or complete washing out of the approach embankment by overflow.
- Dislodging failures or damage to bridge superstructures by the action of buoyancy, flowing water, ice or debris (refer to OHBDC, 1991).
- Bending of piles by the impact of ice or debris.
- Erosion of fill in spill through bridge openings.
- Increased flooding of upstream property due to bridge backwater.
- Increased erosion of downstream banks due to concentration of the flow from the flood

plain.
- Debris or ice jamming caused by low soffits or excessively short spans.
- Erosion of the approach embankment due to shifting of the stream channel.
- Problems caused by inadequate deck drainage systems.

Span Arrangement

The following factors should be considered in selecting a span arrangement, bearing in mind structural limitations and requirements. Care must be taken that the extra cost of meeting horizontal clearance requirements is fully justified by the benefits received, bearing in mind that an increase of span may affect the type of bridge, and therefore the cost.

- In channels subject to severe ice jamming the span lengths should be as large as is economically feasible.
- Short spans (e.g. 5 m or less) should be avoided on streams carrying medium or large debris. The use of larger spans may also discourage beavers building dams at bridges.
- For navigable streams, requirements of the authority having navigational jurisdiction over the stream should be ascertained.

Pier Details

Careful attention to pier design will minimize the danger of scour and other hydraulic problems. The following points should be considered as well as those relating to scour.

- Preliminary pier locations may need adjustment after the foundation investigation has been received.
- A semi-circular vertical pier nose is reported to be the best for minimizing debris problems. A slight positive slope on the pier nose is also acceptable.
- Where large flows enter the bridge opening from the flood plain, end piers should be kept well away from abutments and fill slopes. This approach can avoid a concentration of scour.
- Piles should be protected against impact by ice floes or heavy debris.
- Pier shaft and footing widths (including sheet piles) should be minimized.
- Scour protection for pier and abutment foundations should be considered.

Abutments

- Abutments should normally be aligned with the flood flow and foundations should have ample scour protection.

Superstructures

- At many crossings there is a definite probability or risk that the superstructure will be submerged at some time in its lifetime. For superstructures liable to submergence, explicit design features are required to resist transverse motion (refer to OHBDC, 1991).
- The provision for a suitable clearance from the bridge soffit to the design high water elevation or ice jam level, is important. However, there is often a limit on how high the bridge may be constructed. For instance, limits include cost (i.e. soffit elevation has a direct effect on cost) and the need to keep the approach grade low enough to provide for relief overflow.
- A remedial measure that can be considered for small bridges is to secure the structure to nearby trees with wire cables.

Soffit Elevation

- The height of a bridge relative to flood and ice jam levels is important for safeguarding the superstructure against various forms of damage, minimizing the impacts of the bridge on backwater and navigation; and maximizing the amount of relief flow over the approach grade.
- Where backwater is critical, consideration should be given to maximizing relief overflow.
- At many crossings the soffit elevation may be based on the minimum vertical clearances, but at others the hydraulic requirements may have to be overruled by less flexible factors such as site topography, highway geometry, elevations of adjacent properties or foundation constraints.
- Consider increased clearance where ice or debris conditions are exceptionally severe, and for long spans, especially of light construction.
- Consider reduced clearance:
    - for a curved soffit (e.g. concrete arch or rigid frame) where there is no danger of damage from water, ice or debris;
    - at sites where the height of the approach grade is limited by subsoil conditions;
    - on seasonal highways or highways having an exceptionally low traffic volume;
    - where the lowest point of the soffit is well out of the main channel; and
    - at low-water bridges.

Applicable Hydrologic Methods for Bridge Crossing Design

- For a brief discussion on the application of hydrologic computational procedures, refer to Appendix 3A. For a detailed discussion, refer to Chapter 8.
- Stream flow rates can be determined using the methods in Appendix 3A.
- Peak flow methods, such as rational or modified index flow method can be used for initial or

preliminary bridge opening sizing (i.e. to determine slope, cross-sectional area or lining).

- Peak flow methods are appropriate for final design, but they must be applicable.
- For complex situations, where a more accurate assessment of flow is needed, hydrographic methods may be more appropriate.
- In finalizing a bridge design, a hydraulic backwater assessment should be completed. Generally, such an assessment will include a regulatory flood line calculation which will require that a hydrographic method be used to determine the peak flow input.
- Hydrograph storage routing can be completed at the final design stage to check for any attenuation effects on hydrograph characteristics such as peak flow, peak flow timing, and total duration of the hydrograph.
- Computer programs that can be used to determine runoff hydrographs and also complete hydrograph routing, are presented in Appendix 3A, Table 3A.3.

Applicable Hydraulic Methods for Bridge Crossing Design

- For a brief discussion on the application of hydraulic computational procedures, refer to Appendix 3B. For a detailed discussion, refer to Chapter 8.
- Preliminary sizing of the waterway opening can be completed using methodology presented in Appendix 3B. At final design, the culvert must be sized using exact information.
- Vertical drops, erosion protection, scour protection or flow control structures can be sized using the method presented in Chapter 5.
- Any design of a stream modification must be checked with a detailed backwater assessment that will incorporate any changes to stream channel and profile configurations, as well as any vertical drops, erosion protection, scour protection or flow control structures that may be included as part of the design.
- The appropriate computational backwater method will depend on the complexity of the design as well as the flow regime. Refer to Appendix 3B for a discussion on different applicable methods.
- Computer programs that can be used for hydraulic computations, are presented in Appendix 3B, Table 3B.1.

Information Sources and Working Details for Bridge Crossing Design

In the design of bridge crossings, the practitioner will:

- ensure relevant design considerations are incorporated into the design details;
- use appropriate computational methods to evaluate the performance of the proposed design; and
- ensure that sufficient and appropriate data and assumptions are used.

This can be accomplished by applying the procedures presented in Chapter 5 in conjunction with the previous sections on design considerations and computational procedures. More specifically, Chapter 5 provides details on the design of the following items:

- flow conveyance an backwater;
- river ice;
- scour;
- fish passage;
- debris flow;
- energy dissipation;
- erodible channels;
- stream channel lining material; and
- the hydraulic design of fish habitat structures.

Completing a Culvert Crossing Design

The design of a culvert crossing structure must integrate structural and foundation considerations with the hydraulic considerations of the culvert opening. This section will present the hydraulic considerations for culvert crossings, under the following headings:

- considerations at this stage of the culvert crossing design;
- applicable hydrologic methods for culvert crossing design;
- applicable hydraulic methods for culvert crossing design; and
- information sources and working details for culvert crossing design.

In conjunction with this section, review the section, Factors to Review When Considering a Culvert Crossing.

Considerations at this Stage of the Culvert Crossing Design

The design considerations for culvert crossings include the following:

- common hydraulic problems;
- open footing versus closed invert culverts;
- culvert material;
- culvert shape;
- multi-barrel culverts;
- culvert profile;
- culvert length;
- culvert safety concerns;
- fish passage design flow;
- clay seals; and
- other considerations.

Common Hydraulic Problems

The designer should be aware of the types of problems that culverts are susceptible to, in order to minimize their occurrence in the future. The more common or serious problems are listed below:

- overtopping and washing out of embankments;
- scour at open footing culverts;
- undermining of footings due to channel degradation;

- undermining due to artificial deepening of channel;
- erosion of fill at inlet;
- uplift of corrugated steel culvert inlets;
- invert buckling of corrugated steel culvert;
- outlet erosion;
- debris blockages;
- culvert icing; and
- washouts resulting from percolation through fill.

Open Footing Versus Closed Invert Culverts

- A disadvantage of open footings is that the size of culvert required for safe performance is frequently larger than the equivalently sized closed invert culvert. Open footing culverts are also vulnerable to failure caused by scour, degradation or artificial deepening.
- The use of open footing culverts is recommended at sites where: future deepening is uncertain, such as on some municipal drains or urban drainage schemes; and on fish migration routes where the increased roughness of a natural stone bed is desirable for reducing the flow velocity. However, in the latter case, the same effect may sometimes be achieved in closed invert culverts by placing a layer of stone or gabions, provided that the material will not be moved by the design flood.
- Other than in the circumstances outlined above, open footing culverts should be used only on bedrock or equally scour-resistant material, or if protected by a permanent floor slab with cutoffs.

Culvert Material

The choice of material for culverts depends on the initial cost, service life, maintenance costs, hydraulic performance, ease of construction, salvageability, structural strength, fish passage requirements, and other factors. Materials for culvert construction include corrugated steel, concrete, and plastic (where the choice of culvert material is not critical to the hydraulic design, the Ministry may allow a contractor to bid on alternative materials). Some of the factors to be considered in making a suitable choice are as follows.

- Steel and plastic have the advantage of simpler and quicker construction, particularly in remote areas, while steel has the added advantage of often being at least partly salvageable after being washed out.
- A well designed concrete box culvert is extremely durable under a wide range of conditions.
- Precast concrete and smooth-walled plastic pipes provide more efficient inlets than do sharp edged inlets on metal culverts.
- The greater roughness of corrugated interiors may be an advantage for fish passage and for other situations where barrel or outlet velocities must be reduced.

Culvert Shape

- Cross-sectional shape will depend on the height of fill and depth of flow.
- Circular pipes are structurally efficient, readily available, cost somewhat less than other shapes having the same capacity, and may be somewhat less susceptible to total blockage by icing due to their greater height.
- Corrugated steel pipe arches are useful under low highway grades.
- Horizontal ellipses are also useful at low profile sites, and when properly designed are structurally stronger than pipe arches.
- The advantage of rectangular shapes is that their dimensions (i.e. width and depth) can be varied to suit a wide range of site conditions. However, precast concrete culverts may be more preferable in remote areas, or where the time available for construction is limited.
- Arches of reinforce concrete or corrugate steel are useful for low-profile situations, shallow flows and fish passage; but, they require scour protection (i.e. in erodible soils) and better than average foundation support.

Multi-Barrel Culverts

- In general, the use of multi-barrel culverts should be approached with caution.
- Culverts having more than one barrel or cell may be necessary for wide streams having relatively low depths of flow, and for shallow fills.
- Multi-barrel culverts are also useful when fish passage has to be provided. In such cases, only one barrel needs to meet the fish passage requirements. This barrel may be smaller and at a lower elevation; or, alternatively, a transverse sill may be provided across the other barrels to concentrate low flows through the single barrel.
- Where passage is to be provided for humans, cattle, or other animals, one barrel should be place at a slightly higher elevation so that the invert will remain dry for most of the year.
- On debris carrying streams, multi-barrel culverts are more susceptible to blockage.
- On curved stream alignments, the inner barrel is more prone to silt deposition.

Culvert Profile

- The selection of a suitable profile and depth of embedment for a culvert may be influenced by the following considerations:

  - natural slope of the stream;
  - requirements for improved inlet design;
  - barrel and outlet velocities;
  - future deepening of municipal drains;
  - stream bed degradation or aggradation;
  - fish passage;

  - differential settlement of the culvert;
  - undermining of the culvert ends; and
  - deposition of sediment inside the culvert.

- Flexible culverts on compressible soils, especially under high fills, should be longitudinally cambered to overcome the effects of differential settlement.

Culvert Length

- The length of a culvert should be sufficient to keep the fill from obstructing the waterway.
- Excess length should be minimized to avoid fish passage problems, or uplift pressures that may lead to failures. If extra length is required to accommodate future widenings, uplift pressures should be checked.

Culvert Safety Concerns

- Culvert ends projecting from fill slopes may be a hazard to vehicles that run off the road. Culvert ends may be mitred to match the fill slope or shaped to reduce the hazard to vehicles running off the road.
- In urban areas, culverts with significant depth should be fenced around the wingwalls and headwall to reduce the risk of people falling over.
- Grates may be installed to keep children from entering the culvert, and large drops in the stream or culvert bed should be avoided. A grate should only be specified after consideration of all possible negative effects, such as maintenance requirements or possible blockage of the waterway due to debris accumulation.

Fish Passage Design Flow

Culverts on fish migration routes should permit the passage of fish during flow conditions likely to prevail at the time of migration.

Clay Seals

For sites where a significant head differential may exist, a means of reducing the likelihood of piping is to incorporate an impermeable clay seal on the upstream side of the embankments (refer to Chapter 5).

Other Considerations

Other design considerations are presented in Chapter 5. These include:

- tailwater level;
- improved culvert inlet design;
- culvert end treatment; and
- embankment fills adjacent to culverts.

Applicable Hydrologic Methods for Culvert Crossing Design

- For a brief discussion on the application of hydrologic computational procedures, refer to Appendix 3A. For a detailed discussion, refer to Chapter 8.
- Steam flow rates can be determined using the methods in Appendix 3A.
- Peak flow methods, such as rational or modified index flow method can be used for initial or preliminary culvert sizing (ie. slope, cross-sectional area or lining).
- Peak flow methods are appropriate for final design, but they must be applicable .
- For complex situations, where a more accurate assessment of flow is needed, hydrographic methods may be more appropriate.
- In finalizing a culvert design, a hydraulic backwater assessment should be completed. Generally, such an assessment will include a regulatory flood line calculation which will require that a hydrographic method be used to determine the peak flow input.
- Hydrograph storage routing can be completed at the final design stage to check for any attenuation effects on hydrograph characteristics such as peak flow, peak flow timing, and total duration of the hydrograph.
- Computer programs that can be used to determine runoff hydrographs and also complete hydrograph routing, are presented in Appendix 3A, Table 3A.3.

Applicable Hydraulic Methods for Culvert Crossing Design

- For a brief discussion on the application of hydraulic computational procedures, refer to Appendix 3B. For a detailed discussion, refer to Chapter 8.
- Inlet and outlet control should always be checked to determine the governing situation. For initial or preliminary sizing, assumptions can be made to simplify the calculation. At final design, the culvert must be sized using exact information.
- Vertical drops, erosion protection, scour protection or flow control structures can be sized using the method presented in Chapter 5.
- Any design must be checked with a detailed backwater assessment that will incorporate any changes to stream channel and profile configurations, as well as any vertical drops, erosion protection, scour protection or flow control structures that may be included as part of the design.

- The appropriate computational backwater method will depend on the complexity of the design as well as the flow regime. Refer Appendix 3B for a discussion on different applicable methods.
- Computer programs that can be used for hydraulic computations, are presented in Appendix 3B, Table 3B.1.

Information Sources and Working Details for Culvert Crossing Design

In the design of culvert crossings, the practitioner will:

- ensure relevant design considerations are incorporated into the design details;
- use appropriate computational methods to evaluate the performance of the proposed design; and
- ensure that sufficient and appropriate data and assumptions are used.

This can be accomplished by applying the procedures presented in Chapter 5 in conjunction with the previous sections on design considerations and computational procedures. More specifically, Chapter 5 provides details on the following items:

- flow through culverts;
- river ice;
- scour;
- fish passage;
- debris flow;
- energy dissipation;.
- erodible stream channels;
- stream channel lining material; and
- hydraulic design of fish habitat structures.

Completing a Stream Channel Modification Design

Avoiding stream channel modifications is the only sure way to minimize potential impacts and future problems. A thorough search for highway alternatives or options that can avoid or minimize stream channel modification, is an important task and should be done as early in the planning or design stage as possible. These issues are discussed in The Need for Stream Modifications and Adverse Impacts Associated with Stream Modifications, which should be reviewed in conjunction with this section.

Traditionally, any stream channel modification scheme may increase the hydraulic efficiency by straightening, widening, or channelizing the existing stream channel. In highway design, stream channel modification will be required in the following scenarios:

- several stream channels are diverted and combined to form a single stream channel;
- a stream channel is realigned to avoid intersection with the highway;
- a stream channel is modified to accommodate a bridge or culvert;
- an entire stream channel segment is replaced with a culvert or storm sewer; or
- modifications are required to mitigate an on-going erosion problem.

The discussion in this section will focus on the design considerations for stream modifications. This will be covered under the following headings:

- considerations at this stage of the stream modification design;
- applicable hydrologic methods for stream modification design;
- applicable hydraulic methods for stream modification design; and
- info sources and working details for stream modification design.

Considerations at this Stage of the Stream Modification Design

The design components associated with stream channel modifications are covered in Chapter 5. The main considerations are summarized here for convenience.

- Any stream channel modification scheme should consider costs:

  - construction and long term maintenance costs;
  - long term costs to remedy potential environmental impacts; and
  - the relative risks and costs associated with future damages that could result from legal action brought forth by a riparian owner.

- Other professional disciplines, internal and external, should be consulted.

- Storm sewer or highway ditch inlet and outlet locations should be located.
- Any realigned stream channel segment should be reasonably compatible with the natural flood flow pattern.
- The natural character of the stream and its surroundings should be preserved.
- Velocities in the channel and its vicinity should be limited, where necessary, by the provision of check dams or other means, to those allowable for the natural bed and banks. Alternatively, erosion control measures should be provided at critical points.
- Hydraulic calculations and erosion controls (temporary and permanent) should take into account roughness coefficients applicable to conditions both immediately after construction and after the channel has become vegetated and stabilized.
- Side slopes should be appropriate for the bank materials and groundwater conditions.
- Special provision for fish habitat should be made on gravel streams and others containing valuable fish resources.
- The diversion ends should be aligned so as to not adversely affect the upstream or downstream channel or adjacent property.
- Construction should be timed so as to avoid spawning periods.
- Radii of bends should approximate those of the original channel.
- The possibility of non-uniform flow in a short diversion should be considered.
- The use of pilot cuts is not recommended. (These are small artificial meander cutoffs that rely on the river flow to enlarge them to their ultimate size). They can have serious environmental effects arising from erosion and sedimentation.

Applicable Hydrologic Methods for Stream Modification Design

- For a brief discussion on the application of hydrologic computational procedures, refer to Appendix 3A. For a detailed discussion, refer to Chapter 8.
- Steam flow rates can be determined using the methods in Appendix 3A.
- Peak flow methods, such as rational or modified index flow method can be used as input to an appropriate hydraulic method for preliminary stream channel modification sizing (i.e. slope, cross-sectional area or lining).
- In finalizing a stream channel modification design, a hydraulic backwater assessment should be completed. Generally, such an assessment will include a regulatory flood line calculation which will require that a hydrographic method be used to determine the peak flow input.
- Another hydrologic consideration in stream channel modification design is hydrograph routing. Hydrograph routing should be completed at the design stage to check for the effects of the proposed stream channel modification on hydrograph characteristics such as peak flow, peak flow timing, and total duration of the hydrograph.
- Computer programs that can be used to determine runoff hydrographs and also complete hydrograph routing, are presented in Appendix 3A, Table 3A.3.

Applicable Hydraulic Methods for Stream Modification Design

- For a brief discussion on the application of hydraulic computational procedures, refer to Appendix 3B. For a detailed discussion, refer to Chapter 8.
- Manning equation or a simple standard step method application are quick and efficient ways to complete preliminary sizing of stream channel modifications. Refer to Appendix 3B.
- Vertical drops, erosion protection, or scour protection can be sized using the methods presented in Chapter 5.
- Supercritical conditions should be checked.
- Any design of a stream channel modification must be checked with a detailed backwater assessment that will incorporate any changes to stream channel and profile configurations, as well as any vertical drops, erosion protection, scour protection or flow control structures that may be included as part of the design.
- The stage-storage relationship for the modified stream channel and flood plain should be similar to the stage-storage relationship that exists for the natural conditions.
- The appropriate computational backwater method will depend on the complexity of the design as well as the flow regime. Refer to Appendix 3B, for a discussion on different applicable methods.
- Computer programs that can be used for hydraulics computations, are presented in Appendix 3B, Table 3B.1.

Information Sources and Working Details for Stream Channel Modification Design

In the design of stream channel modifications, the practitioner will:

- ensure relevant design considerations are incorporated into the design details;
- use appropriate computational methods to evaluate the performance of the proposed design; and
- ensure that sufficient and appropriate data and assumptions are used.

This can be accomplished by applying the procedures presented in Chapter 5 in conjunction with the previous sections on design considerations and computational procedures. More specifically, Chapter 5 provides details on the design of the following items:

- remedial erosion control works;
- suitable stream channel sections;
- type of lining material, vegetative or otherwise;
- stream channel bends, meanders and alignments;
- energy dissipators;
- scour protection; and
- the hydraulic design of fish habitat structures and fish passage.

Developing a Surface Drainage Design

The considerations for developing a surface drainage design that are presented in this section, are best suited for the preliminary stages of the highway planning and design process. If the highway project is at the later stages of design, refer to the following sections.

- Completing a Storm Sewer Design;
- Completing a Roadside Ditch Design; or
- Completing a Major Drainage System Design.

In developing a highway surface drainage design, consideration must be given to the following components.

- **The minor drainage system** collects runoff that results from the more frequent storm events (2 yr to 10 yr range), and conveys the runoff to the outlet at the receiving system. For an urban highway, the minor drainage system usually consists of curbs, gutters, catchbasin inlets, storm sewers, minor swales and roadside ditches. For a rural highway, the minor drainage system generally consists of roadside ditches and minor swales. It can also include gutters and catchbasin inlets; however, these components are not frequently used in rural highways.
- **The major drainage system** is the route that is followed by runoff when the capacity of the minor drainage system is exceeded. The major drainage system consists of:
    - the highway surface, median drains, boulevards, and storage areas within the right-of-way;
    - swales, channels or roadside ditches conveying the major storm runoff away from the right-of-way; and
    - the receiving streams, channels, ravines, trunk storm sewers or ponds.

When developing the surface drainage system, the practitioner needs to fit the surface drainage options to the highway alternative. This can be achieved by:

- reviewing the various considerations for developing the surface drainage design;
- selecting the surface drainage option(s); and
- reviewing the possible considerations related to the highway.

A summary of the tasks is presented in Table 3.3.

Table 3.3: Fitting the Surface Drainage Options to the Highway Alternative

Task	Considerations for Developing the Surface Drainage Design	Possible Surface Drainage Options	Possible Highway Considerations
Develop the Minor System	• selection of the minor system	• roadside ditch • storm sewer	• highway alignment • highway profile • highway cross-section
Develop the Major System	• check the major system • check the receiving system	• the highway surface profile, median drains, boulevards, and storage areas within the right-of-way • swales, channels or roadside ditches conveying the major storm runoff away from the right-of-way the receiving streams, channels, ravines, trunk sewers or ponds	• highway alignment • highway profile • highway cross-section

The latitude with which the practitioner has in completing these tasks will also depend on two other factors.

- **The highway project** can be grouped as new highways, modifications to an existing highway, or a rehabilitation (refer to Figure 3.3). For new highways, the practitioner may have more flexibility in selecting surface drainage options, as different highway alignments, profiles and cross-sections may be under consideration. Using the project objectives and criteria as a guide, the form of the minor drainage system, and the major drainage system route may all be adjusted to suit the different highway alternatives that may exist. In contrast, the practitioner may not have the same flexibility for a modification to an existing highway. For instance, in the case of widenings, the form of minor drainage system, and the major drainage system route may already be set. In such a case, there is very little that a practitioner can do with respect to developing the design; so, the project will likely focus on design specifics.
- **The planning and design process** may include various stages, as presented in Figure 3.1. The flexibility that a practitioner has in developing a surface drainage design will be greatly influenced by the stage in which the planning and design process has progressed to, when the surface drainage system is first considered. To properly develop surface drainage designs, it is critical to consider surface drainage as early on in the planning and design process as possible, preferably at the study options or the preliminary design stages rather than at the later stages of design.

It is also important to recognize that the planning and design process is iterative. As the project evolves through the various stages and steps, the surface drainage design should be evaluated to ensure compliance with the objectives and criteria. A re-adjustment of the surface drainage design should occur whenever it has been determined that the objectives or criteria have not been fully complied with.

The following subsections present the various considerations to be applied in developing the surface drainage design:

- selection of the minor drainage system (roadside ditch or storm sewer, gutters, inlets, outlets);
- advantages of storm sewers as compared to roadside ditches;
- advantages of roadside ditches as compared to storm sewers;
- checking the major drainage system;
- checking the receiving system; and
- considering data needs.

Selection of the Minor Drainage System

The highway cross section will determine the physical layout of the minor drainage system. There are two basic highway cross sections. Each consists of the highway surface and:

- curbs, gutters, catchbasins and storm sewers; or
- shoulders and roadside ditches.

Advantages of Storm Sewers as Compared to Roadside Ditches

- Less land is required. In contrast, roadside ditches will generally have relatively fat slopes (i.e. 2h:1v or flatter). As the ditch invert deepens, more land surface area will be needed to accommodate the ditch width.
- There is more flexibility in the selection of suitable inverts, slopes and diameters as the storm sewer is not limited by grading limitations, as is the case for roadside ditches.
- Storm sewers may be more economical in highway cut sections where groundwater flows may need to be intersected.
- Storm sewers are generally more economical for relatively small discharges.
- Storm sewers are more feasible where grades are steep and ditch erosion may be a problem.
- Storm sewers may be safer as open ditches will be a hazard to errant vehicles.
- Roadside ditches may accumulate garbage resulting in a poor appearance.

Advantages of Roadside Ditches as Compared to Storm Sewers

- Roadside ditches will generally cause less environmental and drainage-related impacts. For instance, ditches can reduce runoff volumes and peak flows as the ditches will attenuate flows and allow for infiltration.
- Roadside ditches can be more readily modified than can storm sewers; so, they can be more

suitable for highways which may be widened later.

- Roadside ditches can be designed to convey major flows; however, storm sewers are generally not a feasible or economically viable solution for major storm conveyance.

Checking the Major Drainage System

- The major drainage system always exists, even if it is not planned or designed. If a highway disrupts the natural major drainage system and no reprovision is made, the major flow will be forced to find its own way to go downstream. The result could then be hazardous.
- The major drainage system must provide a continuous overland route for the severe storm runoff events (i.e. up to the regulatory storm) that cannot be conveyed by the minor drainage system (i.e. 10 yr up to the regulatory storm).
- The major drainage system must be checked to ensure that it is not inadvertently cut off by highway profiles, median barriers or noise barriers; and, that it can convey the major storm, which is the greater of the 100 yr or the regulatory storm.
- Overflow routes from road sags to the receiving system must be provided to ensure that water does not pond to excessive depths on the highway surface.
- The major drainage system should be checked to prevent undue hazards to the public and damage to property adjacent to the highway. Consult the appropriate offices regarding requirements for emergency access.
- The highway must be checked to ensure that it can remain accessible to emergency vehicles during major storm events unless such access is not required.

Checking the Receiving System

If the discharge from a surface drainage system is likely to significantly increase erosion or affect the water quantity or quality of the receiving system (i.e. stream, channel, storm sewer, etc.), consideration should be given to the following:

- protecting the receiving system through the application of instream erosion control measures; or
- controlling the runoff and applying appropriate stormwater management and best management practices to mediate any potential impacts.

For a detailed discussion on instream works refer to the section on Developing a Water Crossing Design. For a detail discussion on best management practices, refer to the section, Developing a Stormwater Management Design.

Considering Data Needs

For completeness, a brief discussion is included on data that will be useful in the development of a surface drainage system.

- Details of existing drainage systems, including incoming sewers and ditches.
- Existing drainage problems which could be aggravated or can be relieved by the proposed works.
- Potential drainage problems that might be created by the proposed works.
- High water or floods experienced along the route, including flooding from watercourses.
- Proposed outfall location(s).
- Details and cross sections of receiving channels, storm sewers and overland flow routes.
- Adequacy and condition of existing watercourses or storm sewers, to act as receiving systems.
- Details of depression areas having no surface outlet.
- Possible drainage route of major drainage system.
- Properties which may be affected and any existing flooding or erosion problems.

Completing a Storm Sewer Design

The discussion in the section will focus on the design considerations for storm sewers. This will be covered under the following headings:

- considerations at this stage of the storm sewer design;
- applicable hydrologic methods for storm sewer design;
- applicable hydraulic methods for storm sewer design; and
- information sources and working details for storm sewer design.

Considerations at this Stage of the Storm Sewer Design

- The storm sewer system must have a sufficient outlet.
- The storm sewer system is designed to convey the more frequent runoff events, generally up to the 10 yr design flow.
- The catchbasin inlet spacing should be checked to ensure that the spread flow in the gutters conforms with design criteria.
- The storm sewer diameter, slope and roughness should be selected to ensure that the storm sewer can convey the appropriate design flow rate. Velocities should be checked for pipe abrasion.
- A hydraulic grade line assessment should be completed to check for surcharging where: the capacity of a storm sewer system is not known; or a backwater potential exists at the outlet; or there is a concern that the capacity of a receiving storm sewer system cannot accommodate the required design flow.
- Any impacts to a receiving stream or channel should be assessed. Impacts may be in the form of erosion at the outlet, and changes to stream channel hydrograph characteristics.
- Provide proper connections for the storm sewers with the receiving drain or outlet to a receiving stream.
- Ensure that existing outlets are reconnected to the sewers.
- Ensure that the outlets of pavement subdrains that are connected to the sewers, will not be submerged under all design flow conditions.

Applicable Hydrologic Methods for Storm Sewer Design

- For a brief discussion on the application of hydrologic computational procedures, refer to Appendix 3A. For a detailed discussion, refer to Chapter 8.
- Design procedures use the rational method (refer to Appendix 3A). See Chapter 4 for detailsof sign procedures

- The MTO Rational Drainage Model can be used to facilitate peak flow derivation.
- Hydrographs of highway surface runoff are generally only required where surcharge conditions may be a concern and a hydraulic grade line assessment is required. OTTSWMM is the only computer program presented in Table 3A.3 (see Appendix 3A) that is capable of separating highway surface flow into major and minor hydrographs. The minor flow hydrograph is then input into EXTRAN to determine the hydraulic grade line, total depth of flow and a time-history output of velocity. OTTSWMM does not have a surcharge analysis capability.
- If there is a concern that the storm sewer discharge may have an adverse impact on a receiving stream channel, hydrograph routing should be completed at the final design stage to check for any effects on the receiving stream or channel hydrograph characteristics (e.g. peak flow rate, peak flow timing, and total duration of the hydrograph).
- Computer programs that can be used to determine runoff hydrographs and complete hydrograph routing, are presented in Appendix 3A, Table 3A.3.

Applicable Hydraulic Methods for Storm Sewer Design

- For a brief discussion on the application of hydraulic computational procedures, refer to Appendix 3B. For a detailed discussion, refer to Chapter 8.
- The storm sewer design procedure uses Manning equation (refer to Appendix 3B). Chapter 4 provides details on storm sewer sizing.
- EXTRAN does have the capability to complete a detailed dynamic surcharge analysis. Minor drainage system hydrographs can be imported from OTTSWMM.
- The MTO Storm Sewer Model provides a simple means of verifying a design or conducting a backwater assessment. It is a static model, that is an assessment is completed for one peak flow rate only. Storage effects are ignored.
- If there are concerns with regards to the water level in a receiving stream or channel, water surface profiles should be checked (refer to Appendix 3B for more details).

Information Sources and Working Details for Storm Sewer Design

In the design of storm sewers, the practitioner will ensure that relevant design considerations are incorporated into the design details; and that computational methods will be used to evaluate the performance of the proposed design.

This can be accomplished by applying the procedures presented in Chapter 4 in conjunction with the previous sections on design considerations and computational procedures. More specifically, Chapter 4 provides details on the design of sewer material and sewer elevations, hydraulic considerations (diameter, slope, velocity, roughness), sewer inlet and outlet conditions, exfiltration to local subgrade, safety for people and vehicles, and sewer accessories.

Completing a Roadside Ditch Design

The discussion in this section focuses on the design considerations for roadside ditches functioning as a minor drainage system. This will be covered under the following headings:

- considerations at this stage of the roadside ditch design;
- applicable hydrologic methods for roadside ditch design;
- applicable hydraulic methods for roadside ditch design; and
- information sources and working details for roadside ditch design.

Considerations at this Stage of the Roadside Ditch Design

- The roadside ditch system and the storm sewer system have the same basic design criteria; the ditch is designed to convey the more frequent runoff events, generally up to the 10 yr design flow. However, in some cases, the roadside ditch may be designed to convey major flows (i.e. flows in access of the minor flow up to the 100 yr or the regulatory storm, whichever is the adopted design criterion), in addition to minor flows.
- For roadside ditches that will function as a major drainage system, see the section Completing a Major Drainage System Design.
- The ditch cross-sectional surface area, bottom slope, side slope and surface roughness should be selected to ensure that the ditch can convey the appropriate design flow rate. Flow velocities should be checked for erosion and scour. The water surface profile should be checked to ensure conformance with safety requirements.
- Procedures for stream channel modifications explained in this chapter and Chapter 5, can be applied to roadside ditch design.
- If the bottom slopes are too steep and the corresponding velocities too high, vertical drops should be introduced (see Chapter 5).
- Where the capacity of a roadside ditch system is not known, a backwater potential exists, or there is a concern that the capacity of the roadside ditch system will not meet the required design flow, a backwater analysis and/or a energy grade line analysis, should be completed to determine the water surface profile.

Applicable Hydrologic Methods for Roadside Ditch Design

- For a brief discussion on the application of hydrologic computational procedures, refer to Appendix 3A. For a detailed discussion, refer to Chapter 8.
- Flow rates can be determined using the methods in Appendix 3A.
- The Rational Method can be used to proportion slopes and cross-sectional area, and for

selecting lining materials.
- The MTO Rational Drainage Model can be used to facilitate peak flow derivation.
- If there is a concern that the roadside ditch discharge may impact the receiving stream channel flow, hydrograph routing should be completed at the design stage to check for any effects on the receiving stream channel hydrograph characteristics (e.g. peak flow rate, peak flow timing, and total duration of the hydrograph).
- Computer programs that can be used to determine runoff hydrographs and also complete hydrograph routing, are presented in Appendix 3A, Table 3A.3.

Applicable Hydraulic Methods for Roadside Ditch Design

- For a brief discussion on the application of hydraulic computational procedures, refer to Appendix 3B. For a detailed discussion, refer to Chapter 8.
- The Manning equation can be used for design purposes.
- If a backwater analysis is required a simple method is the standard step method.
- Another simplified design method is MTO computer program CHANDE.
- If a detailed assessment of water levels and flow (i.e. a backwater analysis), the computer program HEC2 (see Table 3B.1) can be applied.

Information Sources and Working Details for Roadside Ditch Design

In the design of roadside ditches, the practitioner will:

- ensure relevant design considerations are incorporated into the design details;
- use appropriate computational methods to evaluate the performance of the proposed design; and
- ensure that sufficient and appropriate data and assumptions are used.

This can be accomplished by applying the procedures presented in Chapter 4 in conjunction with the previous sections on design considerations and computational procedures. More specifically, Chapter 4 provides details on the design of the following items:

- ditch cross section and lining;
- profile, invert and crest elevations;
- very flat terrain;
- very steep terrain;
- roadside safety;
- porous soil;
- limited availability of right-of-way;
- entrances to adjacent property; and
- aesthetic considerations.

Completing a Major Drainage System Design

This section presents design considerations for major drainage systems. This will be covered under the following headings:

- considerations at this stage of the major drainage system design;
- applicable hydrologic methods for major drainage system design;
- applicable hydraulic methods for major drainage system design; and
- information sources and working details for major drainage system design.

Considerations at this Stage of the Major Drainage System Design

- The overland flow route to the receiving drainage system (stream, channel, storm sewer, etc.), must be inspected in the field to ensure that the surface flow has an uninterrupted path that will not cause damage to local property.
- Ditches, channels and sewers which are part of the major drainage system, are designed for the 100 yr flow or the regulatory storm flow, which ever is the adopted design criterion.
- Flow along highways or roadways should be kept within reasonable limits during major storms, to permit the passage of emergency vehicles, and for safety considerations, in accordance with current design criteria.
- A suggested procedure for checking the performance of the major drainage system is to identify the flow route. Then consider the flow conditions on the surface at critical locations such as grades, drainage of sags, and the flow path from the sags. As a final step, consider the need for erosion control measures.
- Sags in the highway profile are the areas most affected by major storm flow. When the runoff exceeds the design capacity of the minor drainage system, the carryover flow will pass inlets on the approach grades, and accumulate until a sag is reached. At this point, a suitable relief outlet must be provided. If there is no possibility of a suitable relief outlet, the situation should be studied, and a suitable drain, such as a major flow ditch, should be provided to intercept the flow and bring it to a sufficient outlet. The lack of an adequate outlet may create a drainage hazard.
- The lining material for swales and channels should withstand the major storm velocities and tractive forces.

Applicable Hydrologic Methods for Major Drainage System Design

- For a brief discussion on the application of hydrologic computational procedures, refer to Appendix 3A. For a detailed discussion, refer to Chapter 8.
- Flow rates can be calculated using the rational method or a suitable a hydrographic method. Refer to Appendix 3A for a discussion of the rational and hydrographic methods.
- OTTSWMM is the only computer program presented in Table 3A.3 that is capable of separating flow into major and minor hydrographs. It is capable of routing the major flow along the highway surface to determine the total depth of flow. It will also provide the user with a time-history output of velocity.

Applicable Hydraulic Methods for Major Drainage System Design

- For a brief discussion on the application of hydraulic computational procedures, refer to Appendix 3A. For a detailed discussion, refer to Chapter 8.
- A simpler method of determining the depth of flow, is through the application of the Manning equation (refer to Chapter 8). Velocity can be determined by applying the continuity equation or the Manning equation.
- Other methods of determining velocities or depths of flow are the standard step method, or other computer applications (refer to Table 3B.1)

Information Sources and Working Details for Major Drainage System Design

In the design of the major drainage system, the practitioner should:

- ensure relevant design considerations are incorporated into the design details;
- use appropriate computational methods to evaluate the performance of the proposed design; and
- ensure that sufficient and appropriate data and assumptions are used.

This can be accomplished by applying the procedures presented in Chapter 4 in conjunction with the previous sections on design considerations and computational procedures. The design of channels which require hydraulic energy dissipation is presented in Chapter 5.

Developing a Stormwater Management Design

The considerations for developing a stormwater management design that are presented in this section, are best suited for the preliminary stages of the highway planning and design process. If the highway project is at the later stages of design, refer to following sections:

- Completing a Stormwater Quality Control Facility Design; and
- Completing a Stormwater Quantity Control Facility Design.

The highway stormwater management system comprises the surface drainage system (the conveyance system), and the mitigative measures that may be required for the control of stormwater quantity and quality. The mitigative measures return stormwater quality and quantity to a state that is environmentally acceptable. The acceptability of the quantity and quality of stormwater discharging to a receiving stream or lake will be defined by the project objectives and criteria. In developing the stormwater quality and quantity control objectives and criteria it is essential to involve the water resources engineer, environmental professionals, highway engineers, and relevant regulatory agencies. The mandate for the protection of water quality in the Province of Ontario is with the Ministries of the Environment and Energy, and Natural Resources. Therefore, their concurrence with the approach to managing stormwater is necessary. For this purpose, the most current provincial practices for the control of stormwater quality, have been included in this section.

Developments in the methods and practices for the control of stormwater quality and quantity are constantly evolving as new measures are implemented and assessed. Therefore, it is necessary to consider the current state of the practice and how it may apply to stormwater management for highways. It should be noted that the planning for stormwater quality and quantity management should be an integrated and comprehensive process, and should not be approached as two separate entities. Whenever possible, the development of the stormwater management plan should be done on a watershed or basin basis to account for the cumulative effects of multiple quantity and quality control facilities. It is also important to recognize that water quantity control provides improvements to water quality (the reverse is true as well).

When developing a stormwater management plan, the practitioner needs to fit the stormwater management options to the highway alternative. This can be achieved by:

- reviewing the various considerations for developing a stormwater management design;
- selecting the stormwater management option(s); and
- reviewing the possible considerations related to the highway.

A summary of the tasks is presented in Table 3.4.

Table 3.4: Fitting the Stormwater Management Options to the Highway Alternative

Task	Considerations for Developing the Stormwater Management Design	Possible Stormwater Management Options	Possible Highway Considerations
Develop the Stormwater Management System	• the current approach to stormwater management • stormwater impacts associated with highways • identification of Best Management Practices suitable for the highway • receiving water based quality control criteria • assessment of the need for stormwater quality and quantity control	• Quality control BMPs • Quantity control BMPs	• land availability • locating BMP within the r.o.w. • impact on highway subgrade • maintenance requirements • roadside safety

The latitude with which the practitioner has in completing these tasks will also depend on two other factors.

- **The highway project** can be grouped as new highways, modifications to an existing highway or a rehabilitation (refer to Figure 3.3). For new highways, the practitioner may have more flexibility in selecting stormwater management options, as different highway alignments, profiles and cross-sections may be under consideration. Using the project objectives and criteria as a guide, the stormwater management plan may be adjusted to suit the different highway alternatives that may exist. In contrast, the practitioner may not have the same flexibility for a modification to an existing highway. For instance, in the case of widenings, the stormwater management options will be limited. In such a case, there is very little that a practitioner can do with respect to developing the design; so, the project will likely be focussed on design specifics.
- **The planning and design process** may include various stages, as is presented in Figure 3.1. The flexibility that a practitioner has in developing a stormwater management design will be greatly influenced by the stage in which the planning and design process has progressed too, when stormwater management options are first considered. To properly develop stormwater management designs, it is critical to consider stormwater management as early on in the planning and design process as possible, preferably at the study options or the preliminary design stages rather than at the later stages of design.

It is also important to recognize that the planning and design process is iterative. As the project evolves through the various stages and steps, the stormwater management design should be evaluated to ensure compliance with the objectives and criteria. A re-adjustment of the stormwater management design should occur whenever it has been determined that the objectives or criteria have not been fully complied with.

The following subsections present the various considerations to be applied to develop an effective highway stormwater management system:

- the current approach to stormwater management;
- stormwater impacts associated with highways;
- typical contaminants associated with highway stormwater;
- identification of Best Management Practices suitable for highways;
- receiving water based quality control criteria;
- assessing the need for stormwater quality and quantity control;
- location and layout of the stormwater management system.

The Current Approach to Stormwater Management

The current approach to stormwater management applies a holistic approach to stormwater quantity and quality control. This approach is based on the selection, from a wide variety of control mechanisms, of the most suitable system that would best mitigate the impact of development both locally and on a watershed basis. This approach not only takes advantage of recent developments in quantity and quality control technology and practice, but also emphasizes the adoption of simple common sense principles in stormwater management. Some of these principles include source control, reduction of the development foot print, and conveyance control. New control practices apply quality control mechanisms, such as plant uptake and biodegradation, to stormwater treatment. This wide range of quantity and quality control measures are referred to as stormwater Best Management Practices (BMPs).

The BMP approach is now the mainstream approach to the management of stormwater. However, many aspects affecting the performance of BMPs remain unknown, and are the subject of many research projects.

Stormwater Impacts Associated with Highways

To select an appropriate stormwater management control measure, it is necessary to determine the type of impact this measure will mitigate.

The modification of the flow patterns (e.g. runoff hydrograph distribution) and the deterioration stormwater quality, are two basic impacts associated with highways (refer to Table 2.2 in Chapter 2 for a detailed list of possible impacts). In general, the changes to stormwater quantity and quality due to highway development occurs as a result of the associated changes to the contributing watershed. These changes include reduction of imperviousness, flow concentration into channels and storm sewers, and regrading of the natural topography. These changes result in the following impacts.

Stormwater Quantity Impacts

- Reduced infiltration and increased runoff volume.

- Reduction in the time of concentration resulting in increased peak flow rates.
- Increased flow velocities.
- Reduction of base flow in streams due to reduced infiltration, and flow diversion.
- An increase in the frequency of erosive runoff events that result from typical, highly frequent rain storms (e.g. summer thunderstorms).

Stormwater Quality Impacts

- Sediment transport (i.e. sediment carried by water), as a result of erosion.
- Contaminants are transported from the highway and external lands, to the receiving system.
- Reduction in receiver assimilative capacity for contaminants as a result of a decrease in baseflow.
- Increased runoff water temperature due to an increase in paved area.

Typical Contaminants Associated with Highway Stormwater

Typical contaminants associated with highway stormwater runoff include:

- nutrients (phosphorus, and nitrogen) from fertilizer application; and
- heavy metals (lead, cadmium, mercury, zinc, chromium, or arsenic) from engine and brake wear.

Other contaminants that may be associated with highway stormwater runoff are listed below for completeness:

- organic and chemical oxygen demand from the decomposition and breakdown of organic or chemical matter; and
- oil and grease from vehicles.

Contaminants in stormwater occur in two forms, particulate and dissolved. Particulate contaminants are either granular material, organic or chemical, or insoluble contaminant adsorbed into sediment particles. Particulate contaminants are, therefore, transported with the sediment. Soluble contaminants are in solution and are transported directly by the water. Tables 3.5 and 3.6 provide a description of the different mechanisms for water quantity and quality control. The methods for the control of contaminants in stormwater runoff depends on the type of contaminants and the path they take as they are transported within the natural environment.

The extent of deterioration in stormwater quality as a result of development is not only a function of the increased amount of contaminants, but can also be due to changes to stormwater quantity. For example, the increase in sediment loading may be associated with increased erosion as a result of higher runoff flow velocities.

To mitigate the above impacts, the process of controlling stormwater quality and quantity involves the application of BMPs within the highway surface drainage system; or, at the downstream end of the system (ponds), prior to the runoff entering the stream or lake.

For a more detailed discussion on contaminants, refer to Chapter 8.

Identification of Best Management Practices Suitable for Highways

General Considerations

BMPs consists of "soft" measures and “structural” measures. Soft measures are practices that do not involve the construction of a facility. These practices include measures such as regrading and vegetative buffer strips. Structural measures are constructed facilities (e.g. wet ponds) that provide storage for quantity control, and remove certain types of contaminants before allowing runoff to enter a receiving body of water.

In general, BMPs utilize one or more of the mechanisms outlined in Figure 3.4 for quantity and quality control of stormwater. A drainage option will utilize a combination of these mechanisms to achieve the objectives and criteria established for the highway project. Tables 3.5 and 3.6 provide a brief explanation of the BMP mechanisms. Specifics on the effectiveness of these methods are discussed later in this chapter.

Figure 3.4: Stormwater Quantity and Quality Control

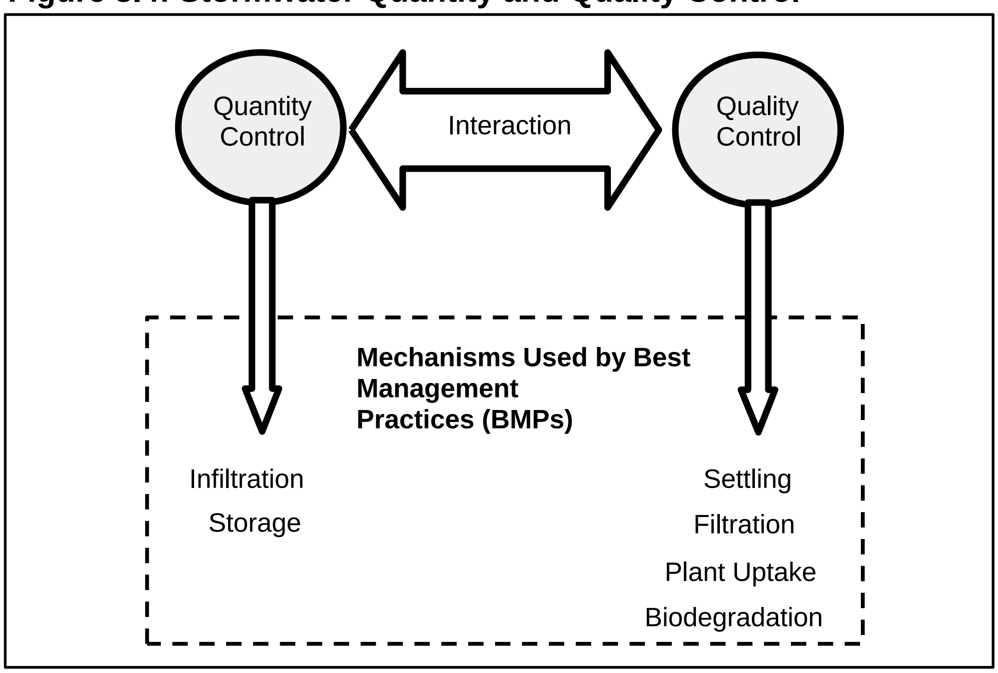


Table 3.5: Water Quantity Control Measures

Control Mechanism	What It Does	Benefit
Infiltration	Reduces surface runoff by infiltrating portion of the runoff into the ground.	**Quantity:** • reduction of the total volume of surface runoff • reduction in the peak flow rate • maintaining of historic low flow in streams **Quality:** • control of stormwater temperature • maintaining stream assimilative capacity due to an increase in low flow
Storage	Controls runoff flow rate by diverting a portion of the flow into storage for future release. (The total volume of flow remains unchanged except for the loss through evaporation from the storage basin surface area).	**Quantity:** • reduction in the peak flow rate • reduction of downstream drainage capacity problem • control of flow velocity • reduction in the need for and/or extent of erosion and sediment control measures **Quality:** • reduction in sediment transport due to erosion • reduction in the transport of contaminants associated with sediment • reduction of sediment through settling

Table 3.6: Water Quality Control Mechanisms

Control Mechanism	What It Does
Settling	Removal of particulate contaminants through settling of sediment.
Filtration	Removal of particulate contaminants through filtration by the soil or filtering material such as geotextile.
Plant uptake	Removal of soluble contaminants, particularly nutrients, through plant uptake.
Biological uptake	Reduction of biological oxygen demand by biodegradation.

The provision of vegetative canopy, and infiltration to increased base flow provide an effective mean of water temperature control.

The Ministry of Environment and Energy (MOEE) has published a document titled *Stormwater Management Practices Planning and Design Manual* (MOEE, 1994), to provide general guidance on planning and design of BMPs. This document has been developed mainly to serve the land development industry. Highway drainage practitioners can refer to it for general guidance on planning matters, as well for BMP design details. This document divides the type of BMPs into three categories, lot level controls, conveyance control and end-of-pipe controls.

BMPs Suitable for Highway Development

Three types of Best Management Practices can be used for highway application:

There are three type of BMPs. They are as follows:

- source control;
- conveyance control; and
- end-of-pipe control.

It is recommended that the priority in applying these BMPs should follow the sequence presented, with end of pipe BMPs applied as the measures of last resort.

Source Control

These practices, for the most part, are soft measures. They include the following.

- **Reduction of chemical applications within the highway right-of-way.** These measures include the controlled use of right-of-way spraying of fertilizers, herbicides, and pesticides, and limiting the application of deicing chemicals, whenever possible. This measure is effective in controlling the production of these chemicals from within the right-of-way. However, their effectiveness is moderated by the transport of chemicals by the highway drainage system from sources outside of the right of way.
- **Street sweeping/catchbasin cleaning.** This BMP has been identified as an ineffective measure. (MOEE, 1994; U.S. Environmental Protection Agency, 1983(b); Versar Inc., 1989).
- **Eliminating direct discharge from surface drainage system.** Direct discharge of the highway runoff from the highway surface to a storm sewer or stream, results in the transport of contaminants directly to a receiver without any mitigation. A typical example of such occurrence is the discharge of bridge deck drainage directly into the stream below. This BMP measure involves directing the runoff from the surface drainage system to a grassed ditch or swale, and allowing it to flow for some distance before entering the receiving water.

Conveyance Control

BMPs for conveyance control includes grassed ditches, vegetated buffer strips, and oil/grit separators. They are used to minimize the potential for scour, and to trap or filter contaminants before the flow enters the surface drainage system.

- **Grassed ditches and swales** has shown to be an effective measure. It provides improvement to runoff quality by filtering suspended sediment and heavy metals within the surface drainage system.

- **Vegetated buffer strips** consist of grass of forested vegetation designed to intercept sheet flow and filter contaminants from the runoff prior to the flow entering the surface drainage system. In general, maintaining vegetation, whether natural or designed, has shown to be an effective, adaptive and flexible measure for highway applications.
- **Oil/Grit separators** is a preliminary treatment device long used in treating sanitary sewage and industrial wastewater where concentrations of oil and grit are high (Steel, E.W., 1960). Examples of appropriate applications are discharges from restaurant kitchens, abattoirs, meat processing plants and oil refinery storage yards. MOEE has noted a study which reported that the performance of oil/grit separators for ordinary urban runoff is poor (MOEE, 1994). As a result, this water quality control measure is not recommended as being suitable for highway application. Their use should be restricted to small areas such as petrol yards, car pool areas and parking lots.

End-of-Pipe Control

End-of-pipe BMPs are sometimes referred to as structural BMPs. Table 3.5 provides a list of the different types of structural BMPs and the associated water quantity/quality control mechanisms. A brief description on the end-of-pipe BMPs that are most often considered for highway drainage management is provided below. The comments on each type serve as a general guide for practitioners to determine whether a given type of BMP may be applicable to their project.

- **Dry ponds** are suitable for stormwater quantity management only. A dry pond detains water during a storm event, and releases it at an outflow rate that is selected to ensure flow related impacts (e.g. flooding) are minimized.
- **Wet ponds** consist of a permanent pool of water which never drains (except during maintenance), and an additional storage space, on top of this pool, to hold the runoff that enters the pond in a storm event. The stored water is gradually released to a receiving water body. The permanent pool provides extended settling time equal to the interval time between storms, and allows the dilution of the discharge during a storm event by mixing the incoming flow with the existing pool of water (clean water). Wet ponds have been found to be very an effective and reliable end of pipe BMP in terms of contaminant removal.
- **Extended dry detention ponds** do not have a permanent pool like a wet pond does. An extended dry pond can only remove solid particles and not dissolved contaminants. The difference between an extended dry pond and a dry pond, is that the release rate of the extended dry pond is selected to mitigate water quality impacts, while the release rate of a dry pond is selected to minimize impacts related to flow (e.g. flooding).
- **Constructed wetland** have been reported to be an effective method of stormwater quality control; but, such reports are few and the sites of successful applications may be in more favourable climatic zones (e.g. the success reported by Martin E.H., 1988 is related to a project located in Florida). Technical literature dealing with the design of wetlands based on scientific principles, are also few and hard to find. Recent publications on wetland design are

notably confined to **natural** wetlands (Marble, A.D., 1991; National Research Council, 1992; Washington State Department of Ecology, 1992). It is prudent not to use constructed wetlands in general practice in highway stormwater quality management until this technique can be evaluated further, and its effectiveness and reliability can be demonstrated beyond doubt. One aspect that may need to be studied if a wetland is built close to a highway is safety: if the wetland is successful and becomes a refuge for birds and small animals, will the movement of these animals create a safety hazard for them as well as for the highway users?

- **Infiltration techniques** include infiltration basins, infiltration trenches and porous pavement. These techniques have a high failure rate both in Ontario and U.S.A. (MOEE, 1994). For instance, they cannot be expected to function during the spring melt period because the subsurface soil will still be frozen at that time of the year. In addition, infiltration techniques have the following basic limitations:

    - techniques are restricted to areas of well-drained soils;
    - infiltration systems commonly fail due to sediment clogging and reduced infiltration. If the system is a subsurface infiltration system, this would require excavation and replacement of the granular material. If the system is a surface infiltration basin, this would require that the basin bottom be scraped/scarified, and the bottom soils replaced in some instances;
    - potential groundwater contamination; and
    - the infiltrated water may weaken the subgrade of the highway pavement.

    Some of the sediment clogging problems may be reduced through design changes. This may include use of sediment traps or head ponds to encourage sedimentation. These changes, however, remove mainly the large sediment particles and have little effect on the small particle sizes that cause clogging. Therefore, infiltration techniques are not recommended for use in highway projects until their effectiveness is substantiated, and, if used, it can be demonstrated that they will not undermine road subgrade and embankments.

Receiving Water Based Quality Control Criteria

Stormwater quality design criteria in Ontario are based on the requirements for the protection of the receiving water quality. In 1991 the MOEE/MNR introduced the *Interim Stormwater Quality Control Guidelines for New Development*. Although these criteria were specifically created for new development, they have been used as a general guide to stormwater management in the province. These criteria provided water quantity control guidelines as a means for water quality control. The criteria required that runoff volumes of 13mm for a warm water fishery, and 25mm for a cold water fishery, be detained for 24 hours. The criteria also encouraged infiltration, source control, and highlighted that end-of-pipe facilities should be used as a means of last resort. However, these criteria were interim and they were expected to evolve as more end-of-pipe facilities came on line and performance monitoring became possible.

A new approach, introduced by MOEE in 1994, is based on the modelling of suspended solids removal efficiency as indicator to contaminant removal efficiency. This approach was based on the *Fish Habitat Protection Guidelines for Developing Areas* developed by MNR in 1993. These guidelines classify the receiver into three levels of fish habitat protection corresponding to a total suspended solids removal efficiency of 80%, 70%, and 60% respectively. A fourth level of protection was added to account for infilling situations.

These criteria may be used as a guide for the design of water quality control facilities. However, the design criteria should be determined in the development of the project objectives and criteria, with the cooperation and involvement of environmental professionals, regulating agencies and the design engineer.

Finally, the following fact is noted: that stormwater quality control is an evolving field and that no formal Provincial Policy is yet in place.

Assessing the Need for Stormwater Quality and Quantity Control

The need for a suitable type of BMP originates from a need to mitigate the local impacts and watershed impacts that may result from highway development. The need should be identified with the aid of the project objectives and criteria, with input from the highway engineer, environmental professionals, regulating agencies, and others involved in water resources management. Existing watershed studies, when available, may provide guidance to the water quality requirements.

It is important to note that structural BMP facilities may require substantial maintenance. Inadequate maintenance of a facility may lead to poor performance. In the worst case, the facility may become a nuisance instead of providing a benefit to the environment. In making a decision on whether or not to use a BMP facility, the potential long-term maintenance requirements should be considered.

Need For Stormwater Quality Control

The need for stormwater quality control should be based on the sensitivity of the receiver to which the highway runoff will drain. The potential impacts associated with the highway project, together with the associated project objectives and criteria outlined in Chapter 2 of this manual, will flag the type of treatment required to protect the receiving watercourse.

It is important to note that water quality control facilities may not be required under all conditions. A level of water quality control can be achieved through source control, modifications to the surface drainage system and from water quantity control facilities, as shown in Table 3.5.

Need For Stormwater Quantity Control

The need for stormwater quantity control is determined by completing a hydrologic analysis of the existing pre-development scenario, and comparing the results to each proposed stormwater management plan that corresponds to each proposed highway development scenario. In most cases, the conveyance system (i.e. a ditch or sewer), will be developed as part of the surface drainage system. The specific form will depend on the configuration of the highway, surface topography, and other factors all of which are driven by the need to safely convey the highway runoff off the right of way to a suitable receiving watercourse.

Location and Layout Plan of the Stormwater Management System

Issues to consider when selecting the location and configuration of BMP facilities are as follows.

- Topography and availability of low lying areas suitable for locating BMP facilities.
- Whenever possible, BMP facilities should be located outside of the flood plain (above the 100-year elevation). Locations should be considered in consultation with the highway engineer, water resources engineers and environmental professionals.
- Availability of land to locate these facilities within the right-of-way and property acquisition issues, should be assessed.
- To confirm that any initial decisions remain to be applicable, this assessment will need to be revisited once the size of the facility has been determined.
- Consideration must also be given to maintenance access. Access should be sufficient to allow for the passage of equipment required for the dredging and removal of sediment from the settling facilities.
- Small storage facilities will have minimal flood control benefit which will diminish as the flood wave travels downstream.
- Multiple storage facilities located in the same drainage basin will affect the timing of the flood waves travelling downstream. This could increase or decrease flood peaks in downstream locations. Primary consideration should be given to the coordination of storage facilities with other drainage structures, on a regional or basin basis.
- It is possible to combine water quality and quantity facilities within a two cell facility, or have separate quality and quantity facilities. The cost of combined facilities is less, however, serious consideration needs to be given to the possibility of flow short circuiting and resuspension of sediment within the facility as high flows pass through.
- The use of separate quality and quantity facilities is recommended. In this case it is also recommended to provide flow splitting upstream of both facilities to minimize resuspension of sediment. A flow splitter redirects the flow in excess of that required for water quality control (i.e. 2-10 year flow), to the water quantity facility. This consideration should be viewed with full awareness that the planning for stormwater quality and quantity management is an integrated process, and should not be approached as two separate entities.
- The facility must fit into the natural setting and landscape design of the site.

Completing a Stormwater Quality Control Facility Design (i.e. Wet Pond, Extended Detention Pond)

The discussion in this section will focus on the design considerations for end-of-pipe stormwater quality control facilities. This will be covered under the following headings:

- considerations at this stage of the quality control facility design;
- applicable hydrologic methods for quality control facility design; and
- applicable hydraulic methods for quality control facility design.

Specific design details on the different components of wet ponds are discussed in Chapter 4. The discussion in this section provides the main considerations for design at the preliminary design stages. For further discussion and detail refer to the publication *Stormwater Management Practices Planning and Design Manual* (MOEE, 1994).

Considerations at this Stage of the Quality Control Facility Design

When designing a BMP facility the following are the main design elements to be considered:

- size;
- length to width ratio;
- detention time;
- inlet and outlet configuration (location, type, capacity, and design of sediment forebay);
- emergency bypass location, type and capacity;
- maintenance access;
- special safety and maintenance requirements;
- grading and planting strategy; and
- other design considerations.

Size

The size of a wet pond refers to such parameters as:

- surface area;
- depth of 1-3 metres can be used as a guide for permanent pools; and
- depth of active storage area.

The size of a stormwater management facility is a function of the following factors.

- **Drainage area of the contributing basin.** Wet ponds require a minimum drainage area to sustain the permanent pool. As a general rule, a drainage area ≥5 hectares (MOEE, 1994) is sufficient to sustain the permanent pool.
- **Precipitation.** The precipitation data required for the design of wet ponds depends on the level of analysis required. There are two levels of analysis used for design of water quality control facilities: the Derived Probability Distribution (DPD) method and continuous simulation. These methods will be discussed in more detail further later on in this chapter. However, for the purpose of selecting the proper precipitation data (i.e. if the DPD design model is used), statistical factors based on long term precipitation records can be used. These factors include:

  - the annual average rainfall;
  - the average event rainfall;
  - the average event duration; and
  - the average interval time between events.

  Where continuous simulation is necessary, the entire precipitation record (i.e. for periods ≥10 years) will be required.
- **The required sediment removal efficiency** will be determined based on the sensitivity of the receiving body of water as discussed in the section on Receiver Based Water Quality Criteria. The sediment removal efficiency required can range from 60-80% depending on the classification of the receiver.
- **The particle size distribution of the suspended sediment** will depend on local sediment conditions and can be determined through sediment analysis or from existing studies, such as watershed studies, if available. The distribution is represented as the percentage by mass of the different particle sizes.

Length to Width Ratio

- It is critical to provide the longest flow path in the pond to minimize short circuiting and maximize the potential for trapping of sediment particles as they settle in the pond. This will improve the removal efficiency of the suspended sediment.
- A ratio > 3:1 can be used as a guide.

Detention Time

- As a general guide, a detention time of 24 hours may be used to allow for sufficient time for the settling of sediment.
- The discharge flow rates should be checked to ensure no serious downstream impact, as

defined by the project objectives and criteria.

- If the outlet is susceptible to clogging, the result will be a reduction in the detention time. This may cause greater discharge flow rates, and greater outlet velocities.

Inlet and Outlet Configuration

- If a large sediment load is expected, the provision of a sediment forebay may be of benefit to trap large particles (particle size >100-150 μm) within a small area that is easier to maintain.
- The greatest benefit of the sediment forebay will be the even distribution of the flow as it moves across the inlet, to the active portion of the pond. This will minimize the potential for short circuiting.
- The design of the outflow control will determine the outlet flow rate and hence the detention time for the pond.
- The outlet may include devices such as weirs, orifice plates, perforated risers, or a combination of them.

Emergency Bypass Location, Type and Capacity

- An emergency spillway should be designed to pass the Regulatory Flood, without failure, under blocked outlet conditions. Reference should be made to the *Technical Guidelines for Flood Plain Management in Ontario* (MNR, 1987) for design criteria related to potential loss of life from dam failure.

Maintenance Access

- Maintenance provisions should be included to ensure access to trash racks, and for removal of sediment.
- Access ramps should be designed to support maintenance equipment.

Special Safety and Maintenance Requirements

- Trash racks and safety provisions to limit flow velocities should be provided on all inlet structures.
- Fencing may be necessary if there is the potential for public access. This should be considered in a case-by-case situation as deemed necessary.
- Roadside safety for errant vehicles should be provided. Consult the highway engineer for further details.

Grading and Planting Strategy

- If public access is possible, grading near the pond edge may be important to ensure safety and to maximize the functionality of the pond.
- When developing a grading and planting strategy, aesthetics should be considered. Consult a landscape professional.

Other Design Considerations

- A minimum freeboard depth of 0.3 m should be allowed between the maximum high water level and the crest of the pond embankment.
- A side slope of 4:1 or flatter should be allowed to provide for maintenance of the pond.
- A geotechnical assessment may be needed to ensure structural stability of the embankments.
- A minimum bottom grade of 1% should be provided to allow proper drainage.

Applicable Hydrologic Methods for Quality Control Facility Design

The type of hydrologic analysis to be applied should depend on the level of risk associated with the downstream impacts. Event simulation may be used for preliminary design. For cases where an accurate prediction of downstream impact is required (i.e. for legal proceeding) continuous modelling may be necessary. For a discussion on the application of hydrologic computational procedures, refer to Appendix 3A. Specific aspects related to quality control facility design are provided below. For a detailed discussion on design applications, refer to Chapter 4. For background theory refer to Chapter 8.

Single Event Simulation

When designing a stormwater quality drainage system, the single event approach is inadequate. This is because of the following three main reasons.

- Stormwater quality management deals with frequent storm events for which removal of contaminants is most effective and most economical. There is no generally accepted method to define such a frequent, single event.
- Stormwater runoff occurs intermittently, not continuously as sanitary sewage and industrial wastewater. A single event cannot account for the effects of this intermittent nature on stormwater quality impacts.
- The parameters of a storm that determine contaminant washoff by runoff, and subsequent removal by a BMP facility, are the volume, rate and duration of flow, and the time interval between storm events. The values of these parameters vary from storm to storm and moment to moment within a storm.

Therefore, when assessing stormwater quality management needs and design results, it is necessary to analyse a long series (e.g. 10 years or longer) of consecutive storm records (Huber, W.C. and R.E. Dickinson, 1988; Loganathan G.V. et al., 1994; Small, M.J. and DiToro, 1979; U.S. Environmental Protection Agency, 1986).

One way to do this is through the use of statistical analysis, such as the Derived Probability Distribution (DPD) method, and another method is continuous simulation.

The DPD Method

- The DPD method assumes that a historical series of storms and contaminants in the runoff can be defined by a set of mathematical equations using statistical theories. By solving these equations, the statistics of the desired output can be obtained to be similar to continuous simulation.
- The DPD method is suitable for use in preliminary design, because of its ease of use and modest data requirements. It can be a powerful computation tool for comparing alternative types of BMP and alternative designs of a selected type of BMP.
- The model MTO SUDS (Adams, 1995), applies the DPD method. It is developed for MTO. Details on the application of this model are provided in Chapter 4.

Continuous Simulation

- Continuous simulation is based on running a hydrologic computer model with the precipitation and other climatic data gathered for the study area over a long period of time (e.g . 10 years or longer), at hourly intervals. The model routes the flows through the BMP facility under design, from the beginning to the end of the storm series. The results are then summarized into a set of statistics showing the expected performance of the facility with respect to the hydrologic conditions during the simulation period.
- This computational method involves a long and tedious process for data entry and calibration. However, the use of continuous simulation may be necessary and has been made more viable with the introduction of faster computer systems. Therefore, continuous simulation is more suitable for detail design.
- The model MTO SWMM extension is developed for MTO to perform this type of analysis. This extension is to be used in conjunction with the generic SWMM developed by USEPA. Details on the application of this model are provided in Chapter 4. Other models are also available that perform continuous modelling. Refer to Appendix 3B for a listing of the models that have been evaluated by MTO.

New models may be developed in the future and models may be available that have not been evaluated by MTO. It is the designer's responsibility to ensure that a suitable model is applied in the design.

The theoretical background of computation methods and computer models is discussed in Appendices 3A, and in Chapter 8. Practitioners may use the information provided as a guide to select suitable computation methods for preliminary and detail design of BMP facilities. For details on the application of any computer models, refer to the model users manual.

Applicable Hydraulic Methods for Quality Control Facility Design

Typically, in highway design applications, reservoir sizing is used to determine the size and configuration of a stormwater detention facility. Reservoir routing is utilized to determine the effect of a reservoir on hydrograph shape, timing and peak. For a discussion on computational procedures, refer to Appendix 3B. Specific aspects related to quality control facility design are provided below. For a detailed discussion on design applications, refer to Chapter 4. For background theory refer to Chapter 8.

Reservoir Sizing

- Reservoir sizing involves three components:
  - the physical configuration of the detention facility (volume and shape);
  - the hydraulics of the outlet structure; and
  - the sediment removal efficiency achieved based on the volume and shape.
- The physical configuration is quantified by determining the volume of storage for various stages of depth or elevation. This relationship is commonly referred to as a stage versus storage representation.
- The hydraulics of the outlet structure is determined by applying basic culvert hydraulics and flow control principles (i.e. orifice controls and weir flow), to determine flow rates at the various stages of depth or elevation. This relationship is commonly referred to as a stage versus discharge representation.
- The sediment removal efficiency is determined by modelling the settleability of the different particle sizes. This can be achieved with the aid of a number of computer model or routines. Two models that provide analysis for sediment removal efficiency are the MTO SUDS and MTO SWMM. A description of these two types of models is provided in the previous section. For detailed information refer to the model users manuals.

Reservoir Routing

The operation of any reservoir or stormwater detention facility, and its effect on the inflow hydrograph, is determined by inputting the reservoir configuration and hydraulic representation into a computer program.

- The computer program will route the inflow hydrograph through the facility and determine the effects of the facility configuration and its hydraulic representation on the outlet hydrograph.
- In the design of stormwater management facilities, routing and sizing are completed simultaneously. The design process is, therefore, iterative.
- Multiple storage facilities located in the same drainage basin will affect the timing of the runoff throughout the conveyance system, which could increase or decrease flood peaks at downstream locations. Consideration should be given to coordinating storage facilities with other drainage structures on a regional or watershed basis.

For further discussion on the theoretical background of hydraulic computation methods and computer models, refer to Appendix 3B and Chapter 8. Practitioners may use the information provided as a guide to select suitable computation methods for preliminary and detail design of BMP facilities. For details on the application of any computer models, refer to the model users manual.

Completing a Stormwater Quantity Control Facility Design (i.e. Dry Ponds)

The discussion in this section will focus on the design considerations for end-of-pipe stormwater quantity control facilities. This will be covered under the following headings:

- considerations at this stage of the quantity control facility design;
- applicable hydrologic methods for quantity control facility design; and
- applicable hydraulic methods for quantity control facility design.

Specific design details on the different components of dry ponds are discussed in Chapter 4. The discussion in this section provides the main considerations for design at the preliminary design stages. For further discussion and detail refer to the publication *Stormwater Management Practices Planning and Design Manual* (MOEE, 1994).

Consideration at this Stage of the Quantity Control Facility Design

When designing a BMP facility the following are the main design elements to be considered:

- size;
- detention time;
- inlet and outlet configuration (location, type, capacity, and design of sediment forebay);
- emergency bypass location, type and capacity;
- maintenance access;
- special safety and maintenance requirements;
- grading and planting strategy; and
- other design considerations.

Size

The size of a dry pond refers to such parameters as:

- surface area; and
- depth.

The size of a stormwater management facility is a function of the following factors.

- **Drainage area.** As a general rule dry ponds require a minimum drainage areas ≥ 5 hectares (MOEE, 1994) to ensure sufficient flow to limit the potential for clogging of the outlet.
- **Precipitation** data required for the design of dry ponds depends on the level of analysis required. If event modelling will be used the precipitation can be provided based on:

  - AES or district IDF curves;
  - an appropriate synthetic rainfall distribution (Kiefer and Chu, SCS, or other) depending on the urban or rural nature of the contributing areas; and
  - any design should be checked using an event with a 24 hour duration.

  The emergency spillway should be designed to convey the regulatory storm. In some cases continuous simulation may be needed. Refer to Appendix 3A for further guidance.

Detention Time

- As a general guide, a detention time of 24 hours may be used.
- The discharge flow rates should be checked to ensure no serious downstream impact, as defined by the project objectives and criteria.
- If the outlet is susceptible to clogging, the result will be a reduction in the detention time. This may cause greater discharge flow rates, and greater outlet velocities.

Inlet and Outlet Configuration

- If both stormwater quantity and quality control is necessary, it is recommended that separate facilities be used. This is to avoid the sediment re-suspension concern associated with combined water quality/quantity control facilities. To achieve this flow splitting is recommended.
- Flow splitting can be provided upstream of the control facilities. This will ensure that the dry pond will function as a water quantity control facility only.
- The inflow rate to the dry pond will be a function of the threshold discharge rate of the flow splitter control device (weir, partial flume, or other).
- The design of the dry pond outlet structure will determine the outlet flow rate and hence the draw down time for the pond
- Outlet structures include devices such as weir, orifice plate, perforated riser, or a combination of these devices. A comprehensive discussion on the different types of outlet devices can be found in the *Stormwater Management Practices Planning and Design Manual* (MOEE, 1994).

Emergency Bypass Location, Type and Capacity

- The emergency spillway should be designed to pass the Regulatory Flood without failure, under blocked outlet conditions. Reference should be made to the *Technical Guidelines for Flood Plain Management in Ontario* (MNR, 1987) for design criteria related to potential loss of life from dam failures.

Maintenance Access

- Maintenance provisions should be included to ensure access to trash racks and for removal of any accumulated sediment.
- Access ramps should be designed to support maintenance equipment.

Special Safety and Maintenance Requirements

- Trash racks and safety provisions to limit flow velocities should be provided on all inlet structures.
- Fencing may be necessary if there is the potential for public access. This should be considered on a case by case situation as deemed necessary.
- Roadside safety for errant vehicles should be provided. Consult the highway engineer for more details.

Grading and Planting Strategy

- If public access is possible, grading near the pond edge may be important to ensure safety and maximize the functionality of the pond.
- When developing a grading and planting strategy, aesthetics should be considered. Consult a landscape professional.

Other Design Considerations

- A minimum freeboard depth of 0.3 metres should be allowed between maximum high water level and the crest of the embankment.
- A side slope of 4:1 or flatter should be allowed to provide for maintenance of the pond.
- A geotechnical assessment should be provided to ensure structural stability of the embankments.
- A minimum bottom grade of 1% should be provided to allow proper drainage of the pond following a storm.

Applicable Hydrologic Methods For Quantity Control Facility Design

The type of hydrologic analysis to be applied should depend on the level of risk associated with the downstream impacts. For preliminary design event simulation may be used. For cases where an accurate prediction of downstream impact is required (i.e. in legal proceedings), continuous modelling may be necessary. For a discussion on the application of hydrologic computational procedures, refer to Appendix 3A. Specific aspects related to quantity control facility design, are provided below. For a detailed discussion on design applications, refer to Chapter 4. For
background theory refer to Chapter 8.

Single Event Modelling

- If a drainage system is required to manage stormwater quantity only, the drainage system can be designed based on a design storm or a series of design storms, for example, 2-100 year, and the Regulatory Flood.
- This approach is satisfactory because the objective of stormwater quantity management is to provide maximum conveyance capacity for the drainage system. Under event simulation, the use of a hydrologic simulation model will be necessary to calculate the inflow hydrograph(s).
- The Rational Method should not be used for storage facility design since the time distribution of the inflow (inflow hydrographs) is required to assess the effect of detention on the outflow time distribution (the outflow hydrograph).
- When using single event modelling 24-hour design storms should be used.
- Refer to Appendix 3B for a complete listing of the different type of single event models reviewed by MTO. Details on the application of one of these models, OTTHYMO, is provided in Chapter 4.

New models may be developed in the future and models may be available that have not been evaluated by MTO. It is the designer's responsibility to ensure that a suitable model is applied in the design.

The theoretical background of computation methods and computer models is discussed in Appendix 3B, and in Chapter 8. Practitioners may use the information provided as a guide to select suitable computation methods for preliminary and detail design of BMP facilities. For details on the application of any computer models, refer to the model users manual.

Applicable Hydraulic Methods for Quantity Control Facility Design

Typically, in highway design applications, reservoir sizing is used to determine the size and configuration of a stormwater detention facility. Reservoir routing is utilized to determine the effect of a reservoir on hydrograph shape, timing and peak. For a discussion on computational procedures, refer to Appendix 3B. Specific aspects related to quantity control facility design, are provided below. For a detailed discussion on design applications, refer to Chapter 4. For background theory refer to Chapter 8.

Reservoir Sizing

- Reservoir sizing involves two components:
  - the physical configuration of the detention facility (volume and shape); and
  - the hydraulics of the outlet structure.
- The physical configuration is quantified by determining the volume of storage for various stages of depth or elevation. This relationship is commonly referred to as a stage versus storage representation.
- The hydraulics of the outlet structure is determined by applying basic culvert hydraulics and flow control principles (i.e. orifice controls and weir flow), to determine flow rates at the various stages of depth or elevation. This relationship is commonly referred to as a stage versus discharge representation.

For further discussion on the theoretical background of hydraulic computation methods and computer models refer to Appendix 3B and Chapter 8. Practitioners may use the information provided as a guide to select suitable computation methods for preliminary and detail design of BMP facilities. For details on the application of any computer models, refer to the model users manual.

Reservoir Routing

The operation of any reservoir or stormwater detention facility, and its effect on the inflow hydrograph, is determined by inputting the reservoir configuration and hydraulic representation into a computer program.

- The computer program will route the inflow hydrograph through the facility and determine the effects of the facility configuration and its hydraulic representation on the outlet hydrograph.
- In the design of stormwater management facilities, routing and sizing are completed simultaneously. The design process is, therefore, iterative.
- Multiple storage facilities located in the same drainage basin will affect the timing of the runoff throughout the conveyance system, which could increase or decrease flood peaks at downstream locations. Consideration should be given to coordinating storage facilities with other drainage structures on a regional or watershed basis.

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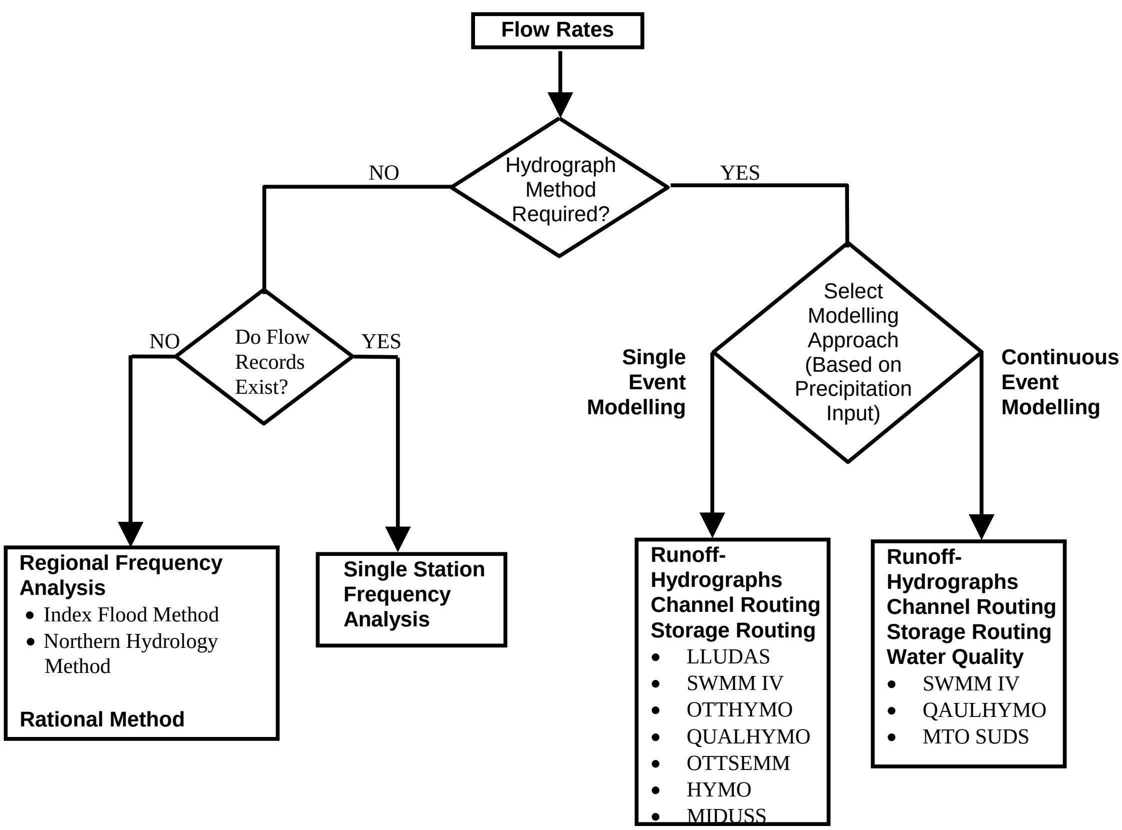
Appendix 3A: Hydrologic Computational Procedures

A summary of the hydrologic computational procedures used to determine flow rates are contained within. The procedures, programs and methodology are guidelines; it is the analyst's/designer's responsibility to recommend and justify the most appropriate methods. For a detailed discussion on the computational methods contained within, refer to Chapter 8.

Hydrologic methodology or analysis, can be represented as follows.

Figure 3A.1: Hydrologic Methodology



Is Hydrograph Simulation Required?

Hydrograph simulation methods are mathematical representations of the response of a surface water system to specific physical processes such as infiltration, evaporation or detention, to produce a graph of runoff plotted against time.

Hydrograph simulation methods are required under the following circumstances:

- the drainage basin is expected to undergo significant urbanization;
- the drainage area will be subject to stormwater management detention or channelization (i.e. hydrograph routing is required);
- the drainage area contains reservoirs and watercourses (i.e. hydrograph routing is required);
- peak flow rates or volumes of runoff will be calculated from a historical rainfall or precipitation event (e.g. Hurricane Hazel or Timmins storm);
- different drainage options are to be tested including regulation (i.e. stormwater management detention, diversion, etc.);
- if land use, times of concentration, or soils conditions vary significantly across the drainage basin; and
- the Rational Method or Modified Index Methods are not applicable.

Hydrograph Simulation Is Required - Select The Modeling Approach

Hydrographic simulation methods are simplified by applying appropriate computer programs or models. Hydrologic modeling is the computer simulation of the response of a catchment area to the input of precipitation, to produce a runoff hydrograph. There are two basic modeling approaches which are fundamentally based on the two forms of precipitation information that are available:

- single event precipitation event; or
- continuous precipitation records.

Chapter 8 presents a more thorough discussion on precipitation inputs and design storms.

Single Event Modeling

Single event modeling is the simulation of the precipitation/runoff process using a short duration precipitation event (i.e. durations ranging from 1hr to a few days).

The precipitation event may be an actual recorded historical storm or a synthetic storm event that is based on a statistical analysis of recorded rainfall. Table 3A.1 compares the different single storm events.

Table 3A.1 Single Storm Events

Type of Single Event	Storm Event	Duration	Time Interval	Land Use[1] Applicability	Degree of Difficulty	
					To Determine	To Apply
Historical	Hurricane Hazel	12 hr or 48 hr	1 hr	urban or rural	low	low
	Timmins	12 hr	1hr	urban or rural	low	low
	Series of Events	Variable	5min to 1hr (as applicable)	urban or rural	medium	high
Synthetic	Chicago (Keifer & Chu)	variable (usually 3hr or 4hr)	variable	urban	medium	low
	SCS Type II	6hr, 12hr or 24hr	15 min	rural	low	low
	AES(30%) -1 hr 12 hr	1 hr 12 hr	5 min 15 min	urban rural	low low	low low
	AES/Hydrotek	1 hr	5 min			low

Note: [1] Urban >20% impervious area , Rural<20% impervious area

Assumptions and Limitations of Single Event Modeling

- The basic assumption is that the return period of runoff from single event models is assumed to equal the return period of the design storm; however, no single event design storm has shown this concept to be valid in all conditions. If an accurate estimate of runoff frequency is needed, continuous simulation may be required.

When Do You Use Single Event Modeling?

- When the peak flow conveyance is the major factor in design (e.g. storm sewer design).
- When the storm event is a designated design storm (e.g. flood line mapping).

How Do You Reduce the Risk Associated with Single Event Modeling?

- By completing a thorough impact assessment that is independent of the design criteria. Table 3A.2 can be applied when determining impacts of events greater than the design criteria.
- Another approach, given a fixed frequency (i.e. 100 yr), is to use storms that have different durations, distributions and intensities. Synthetic storms are most practical for these applications (refer to Table 3A.1).

Table 3A.2: Design Storm Criteria

Drainage Area	Design Criteria	Criteria for Impact Analysis
Less than 100 ha	2 to 50 years	100 year event
	100 years	No additional analysis.
Greater than 100 ha	2 to 100 years	Regulatory Event for all regions where Timmins or Hazel storms apply.
	2 to 50 years	100 years for regions where the 100 year event is the Regulatory Storm.
	100 year	No additional analysis where Timmins and Hazel storms do not apply.

Application of Historical Storm Events

- Hurricane Hazel and the Timmins Storm are historical events; these are storms used in the calculation of the Regulatory Flood.
- Other recorded storm events with the required return frequency and suitable duration can be used (occurs infrequently).

Application of Synthetic Storm Events

- When a recorded storm event with the required return frequency and suitable duration does not exist (i.e. in most cases), synthetics storm are the substitute.
- Synthetic design storms are used to define rainfall distributions for return periods ranging from 2 to 100 years. Generally, the same distribution is used for all return periods and multiplied by the rainfall depth determined from the IDF curves.
- Peak flow rates from urban catchments are usually a function of rainfall intensity rather than rainfall depths. Rural basins usually generate runoff with peaks that correlate well with total rainfall depth.
- The Chicago (Keifer & Chu) design storm is generally applied to urban basins (high percentage of impervious area) where peak runoff rates are largely influenced by peak rainfall intensities. The 24-hour SCS storm is generally applied to undeveloped or rural basins (low percentage of impervious area) where peak flow rates are largely influenced by the total depth of rainfall.

Selecting a Suitable Storm Duration

- Storm duration has traditionally been chosen to be at least equal to the basin time of concentration.
- It is recognized that basin time of concentration will be longer for basins with significant

storage, and that it will vary for each basin and for each precipitation event.

- A storm duration of 24 hours has been chosen arbitrarily for large drainage basins or basins with stormwater management detention or other forms of storage (i.e. ponds, lakes, etc.). A 24-hour duration should be longer than most basin times of concentration for highway drainage applications.
- Where a drainage basin is serviced by a detention facility, a longer duration storm event should be used (i.e. 24hr).

Continuous Event Modeling

Continuous event modeling is the simulation of the precipitation/runoff process using the entire long term meteorological record as input. A frequency analysis is performed on the annual simulated peak flow rates or volumes to determine a frequency curve.

Continuous simulation is expected to generate runoff with a frequency which best approximates reality; however, calibration is required to achieve ultimate accuracy. Typical periods of rainfall data ranges from 10 to 40 years. Continuous simulation can be an expensive, complex and time consuming process, therefore, it is not used frequently for highway drainage design. An alternative to continuous simulation is the simulation of a series of individual historical storm events. Each event is simulated and a frequency analysis of the simulation results is then performed.

Derived Probability Distribution - MTO SUDS

In the design of stormwater quality control management facilities (BMP's), it is necessary to analyse a long series (e.g. 10 years and longer) of consecutive storm records. One way to do this is through the use of the derived probability distribution (DPD) method.

- The DPD method assumes that a historical series of storms and contaminants in the runoff can be defined by a set of mathematical equations using statistical theories. By solving these equations, the statistics of the desired output similar to continuous simulation can be obtained.
- The DPD method is more suitable for use in preliminary design, because of its ease in use and modest data requirement. It can be a powerful computation tools for comparing alternative types of BMPs, and alternative designs of a selected type of BMP.
- There are a number of available computer models that are suitable for stormwater quality management. Table 3A.3 provides a summary of the models that have been evaluated by MTO, and have been found to be suitable for highway design applications in Ontario. In addition to these model, the DPD model MTO SUDS (Adams, 1995), which was developed for MTO, is also available. Details on the application of this model are provided in Chapter 4.

When Do You Use Continuous Event Modelling?

- An accurate estimate of peak flow rate return periods (e.g. during legal proceedings) is required.
- To simulate low flow or base flow conditions.
- For water quality analysis (i.e. pollutograph).

Hydrologic Computer Model Applications

The application of hydrologic computer simulation models to highway drainage design should be undertaken with sound engineering judgement. As a general guide, the Professional Engineers of Ontario offer the following advice when undertaking the design of engineered structures using computer models:

· determine the exact nature of assistance the program provides;
· identify the theory on which the program is based;
· determine the limitations, assumptions, etc. that are included in both the theory and the program;
· check the validity of the program for the intended applications;
· make sure the program is correctly used; and
· verify that the results are correct for each application.

More specifically, the selection of a suitable hydrologic simulation program is dependent on whether the specific computer model suits the application. Factors to be considered when evaluating application include;

- land use compatibility;
- suitable infiltration methodology;
- suitable hydrographic methods;
- hydrologic routing capability;
- Ontario suitability;
- level of effort required; and
- input data requirements.

Some of these factors are discussed in more detail in Chapter 8. Chapter 8 also contains a summary of the appropriate computer models in Appendix 8A. For a quick reference on computer programs that have been previously evaluated by MTO, refer to Table 3A.3. For a complete evaluation, the practitioner should always refer to the user's guide\manual which accompanies each of the programs listed in Table 3A.3.

Table 3A.3: Hydrologic Computer Model Applications

	Single Event					Continuous Event			
Applications	HYMO	OTTHYMO	OTTSWMM	ILLUDAS	MIDUSS	HSP-F	QUALHYMO	STORM	SWMMIV
Land Use: Urban		•	•	•	•	•	•	•	•
Rural	•	•			•	•	•	•	•
Infiltration	•	•	•	•	•	•	•	•	•
Temperature			•			•	•	•	•
Evapostranspiration			•			•	•	•	•
Subsurface Flow						•	•		
Water Balance						•	•	•	•
Water Quality						•	•	•	•
Hydrograph Method	•	•	•	•	•	•	•	•	•
Routing:									
Watercourse\Channel	•	•	•	•	•	•	•		•
Reservoir	•	•	•	•	•	•	•		•
Water Quality						•	•	•	•
Major/Minor System			•						
Receiving Water						•	•		
Ontario Suitability	**Y**	**Y**	**Y**	**Y**	**Y**	**Y**	**Y**	**Y**	**Y**
Level of Effort	**L**	**L**	**M**	**L**	**L**	**H**	**M**	**M**	**M**
Data Requirements	**M**	**M**	**H**	**M**	**M**	**H**	**M**	**M**	**H**

Legend: •-suits application, Y -Yes, L-low, M-medium, H-high

Calibration and Verification of Hydrologic Computer Modelling

Calibration is the process of varying model input parameters until a good fit between measured and simulated values occur.

Calibration and verification is a time-consuming and expensive undertaking as large amounts of data is required to calibrate and verify the model. If the failure of a facility would increase the risk to life or property damage, then data collection for calibration should be considered.

- Continuous simulation models should be calibrated with a minimum of five years of data. Another five years of data should be used to verify the model. Single event models should be calibrated with a minimum of five events.

- Hydrologic simulation models may be calibrated to the results of a single station flood frequency analysis.
- It is also good practice to perform a sensitivity analysis on the input parameters, especially if calibration and verification cannot be undertaken. Sensitivity analyses are usually carried out on parameters which cannot be measured with significant accuracy.

Computer Application Errors

Computer application errors are usually characterized by inaccurate outputs/results or by programs failing to terminate normally. The errors can generally be attributed to the following:

- incorrectly entered data;
- incorrect use of default values intrinsically provided by the program;
- incorrect assumptions made in an application;
- program misapplication;
- poorly written user manuals;
- incorrect interpretation of modelling results; and
- programming errors.

Incorrectly entered data probably accounts for many computer application errors. Incorrectly entered data reduces the designer's efficiency. The designer is encouraged to check data entries before program execution. Another common error is the use of computer programs for conditions beyond the author's intended range of application. Generally, there are conditions which cannot be modelled. It is the designer's responsibility to ensure computer programs are correctly applied.

Hydrograph Simulation Is Not Required and Flow Records Exist

A single station frequency analysis of long-term stream flow records can determine peak flow rates required in the design of highway drainage works.

Single Station Frequency Analysis

Single station frequency analysis is a statistical analysis of a series of recorded stream flow data, obtained from a single gauge station, to determine the specific frequency of occurrence for the sample of stream flow rates.

Application of Single Station Frequency Analysis

- Applicable if no significant upstream land use changes or flow control facilities (i.e. dams),

are proposed, or have occurred, during the period of record (i.e. the basic assumption in the application of a frequency analysis is that the basin to which the results are applied is similar to the watershed from which the flood peaks were recorded).
- Generally, the longer the period of record the greater the confidence in the results (i.e. accuracy is proportionate to the years of record).
- Can be used to validate a hydrologic model.
- A detailed discussion on frequency analysis is contained in Chapter 8. Reference to documentation on the Consolidated Frequency Analysis 88 (CFA88- Environment Canada) and the HYDSTAT (Ontario Ministry of Natural Resources) computer programs can also be made when undertaking a single station frequency analysis.

Hydrograph Simulation is Not Required and Flow Records Do Not Exist

Where flow records do not exist and a hydrograph method is not required, the following methods may be used to calculate peak flow rates:

- Rational Method; or
- Regional Frequency Analysis (Modified Index Flood or Northern Ontario Hydrology Method).

Rational Method

The Rational Method is a computational method that is used to estimate a design flow rate from a catchment area. The rational method calculates the peak flow rate at a particular point due to the runoff contributed from the catchment (drainage) area upstream.

Application of the Rational Method

- The Rational Method is primarily used as a design tool for the design of minor drainage systems such as storm sewers and ditches (refer to Chapter 4 for further details).
- The Rational Method can provide acceptable estimates of peak flow rates in small non-retentive rural watersheds. It is mostly applied in urban applications (i.e. small drainage area) as a design tool to size storm sewers.
- The Rational Method is not suitable for historic rainfall events (Hurricane Hazel or Timmins).
- Present practice in MTO limits its use to watershed drainage areas that are less than 100 ha.
- The applicability of the rational method for rural watershed should be reviewed if there is great variability in soil, vegetation or rainfall.

Regional Frequency Analysis

A regional frequency analysis utilizes regional watershed and climatic characteristics to calculate flow rates. It is most commonly used in watersheds that do not contain stream flow gauge recording stations. Details are discussed in Chapter 8.

Applicable regional frequency analysis methods are:

- the Modified Index Flood Method; and
- the Northern Ontario Hydrology Method.

Modified Index Flood Method

The Modified Index Flood Method is a computational method that was developed from a regional frequency analysis of annually recorded maximum peak flow rates, to produce a statistical regression that calculates a 25 yr runoff event. All other peak flows are calculated by applying a frequency factor to the 25-year value.

- Usually applied to basins where no stream gauge flow records exist.
- The lower limit for the use of the Modified Index Flood method is 25 km^2, as presented in Chapter 8.
- Results should be compared with results from at least one other method.
- Refer to Chapter 8 for further details.

The Northern Ontario Hydrology Method

The Northern Ontario Hydrology Method is a computational method that was developed using a probabilistic/statistical approach, to determine peak flow rates for ungauged streams located in small and medium sized northern Ontario watersheds.

- In small to medium watersheds, the storage in lakes, natural depressions and stream valleys are potentially significant to attenuate the peak flows normally caused by the typical rain and snow melt in the spring.
- Northern Hydrology Method is more fully discussed in *Development of Hydrology Method for Medium-sized Watersheds in Northern Ontario* (Queens University, 1995). A summary of the method is also provided in Chapter 8.

Appendix 3B: Hydraulic Computational Procedures

A summary of the hydraulic computation procedures are contained within. The procedures, programs and methodology are guidelines; it is the analyst's/designer's responsibility to recommend and justify the most appropriate methods. For a detailed discussion on the computational methods contained within, refer to Part 2 and to Chapter 8 (Part 3). Generally, hydraulic computation includes procedures to conduct:

- flow analysis (velocities, water surface profiles, subcritical/supercritical flow);
- hydraulic computer model applications;
- culvert hydraulics (inlet and outlet control);
- reservoir sizing (stage-storage-discharge) and routing;
- stream channel sizing and routing; and
- stream channel stability analysis (tractive force, scour).

Flow Analysis

Typically, in design applications, flow analysis is undertaken to determine the capacity of a particular drainage work. Additionally, flow analysis is used to determine other attributes of flowing water such as water level or surface profiles, velocities, total energy and momentum (force). A brief discussion on the various flow analysis methods is provided below. For a detailed discussion on background theory, equations and computational examples, refer to Chapter 8. For design applications, refer to Part 2.

Manning Equation (Steady, Uniform Flow)

- The Manning equation is a simple and quick method for sizing structures and calculating velocities, or depth of flow. The Manning equation is applicable under steady uniform flow where there are no downstream hydraulic influences (i.e. no backwater effects), and the channel shape, lining material and slope are constant throughout the reach.
- Can be applied to open channels, culverts, bridges and storm sewer applications.
- The MTO computer program CHANDE applies to open channel flow applications.

Open Channel Flow - Standard Step Method (Steady, Gradually Varied Flow)

- Utilizes the principles of continuity and conservation of energy to determine a water surface profile. It is generally applied to open channel flow applications.

- Applies for steady, gradually varied flow conditions where downstream hydraulic influences exist (i.e. backwater effects), and the stream shape, lining material and slope vary throughout the reach.
- An initial depth and flow rate must be known. The method is applicable for subcritical and supercritical flow conditions. Friction losses can be calculated using the Manning equation.
- The method is not applicable for rapidly varied flow conditions because losses due to acceleration cannot be calculated.
- HEC-2 (or HEC-RAS) is a computer program that is based on the standard step method and can be applied to open channel flow, bridge or culvert simulations.

Open Channel Flow - Unsteady Flow

- For unsteady flow conditions, the principles of continuity and conservation of momentum are used to determine water surface profiles and velocities.
- Where predominantly unsteady state flow conditions exist, computer programs such as DWOPPER should be applied. For a detailed list of hydraulic computer programs refer to Table 3B.1.

Open Channel Flow - Subcritical/Supercritical Flow

- When calculating water surface profiles or analysing velocities, subcritical or supercritical flow conditions must always be checked.
- The Froude number, F_r, (refer to Chapter 8) determines which condition is present:

  - if $F_r > 1$, supercritical flow exists (steep slopes);
  - if $F_{r,} < 1$, subcritical flow exists (mild slopes); and
  - if $F_r = 1$, flow is critical.

- For subcritical flow, water surface profiles are calculated from a known downstream point, proceeding in a upstream direction. Flow velocities tend to be low.
- Under supercritical flow conditions, water surface profiles are calculated from a know upstream boundary condition (i.e. cross-section, velocity, slope), proceeding downstream. Flow velocities are high, increasing the potential for erosion.
- Hydraulic jumps will occur at the point where the flow condition changes from supercritical to subcritical. At this transition, turbulent flow occurs thereby increasing the potential for erosion. Whenever the supercritical condition governs, the hydraulic jump should be located. Refer to Chapter 8 for further details.

Storm Sewer Applications - Hydraulic Grade line Analysis

- A hydraulic grade line analysis can be used to determine water surface elevation or pressure head within a storm sewer system.
- Hydraulic grade line analysis applies for steady and unsteady flow conditions where downstream hydraulic influences exist (i.e. backwater effects), which may affect the capacity of the storm sewer system.
- Also applied where there is a concern for storm sewer surcharging, can lead to highway flooding.
- Computations are facilitated through the use computer programs such as EXTRAN or the MTO Storm Sewer Model. See Table 3B.1.

Hydraulic Computer Model Applications

The application of hydraulic computer simulation models to highway drainage design should be undertaken with sound engineering judgement. As a general guide, the Professional Engineers of Ontario offers guidance when undertaking the design of engineered structures using computer models; refer to the Appendix 3A, Hydrologic Model Applications.

- The use of computer programs is recommended where the impact of failure is significant. Although conditions may not warrant the use of computer programs, hydraulic analyses should still employ methods based on design flow conditions.
- Several computer programs have been written to determine water surface profiles for both steady and unsteady gradually varied flow conditions. The programs are written to analyse a given hydraulic condition rather than design according to a set of criteria. Computer programs based on rapidly varied flow conditions or the momentum principle are not readily available.
- Chapter 8 contains a summary of the appropriate computer models in Appendix 8A. For a quick reference on computer programs that have been previously evaluated by MTO, refer to Table 3B.1, which identifies the characteristics of several computer programs that can be used for hydraulic analysis
- The unsteady flow computer programs DWOPER and DAMBRK have been developed for large river systems and may not be applicable to many problems involving highway drainage. Also, EXTRAN has been developed for sewer systems. The designer is encouraged to review these programs and ensure the programs are used in the appropriate circumstances.
- The programs HEC-2 (or HEC-RAS) and WSPRO have been used extensively in the analyses and design of open channels. The programs can be used for both subcritical and supercritical flow conditions, but will not model rapidly varied flow. Applications where it is advantageous to apply HEC-2 (or HEC-RAS) or WSPRO instead of the "by-hand" methods (ie. Manning or Standard Step) are where:

- the reach is complex and requires many cross-sections to properly represent the reach;
- different flow scenarios or stream configurations are to be simulated; or
- a more detailed and accurate assessment is needed.

Computer Application Errors

Computer application errors are usually characterized by inaccurate outputs/results or by programs failing to terminate normally. The errors can generally be attributed to the following:

- incorrectly entered data;
- incorrect use of default values intrinsically provided by the program;
- incorrect assumptions made in an application;
- program misapplication;
- poorly written user manuals;
- incorrect interpretation of modelling results; and
- programming errors.

Incorrectly entered data probably can account for many computer application errors. Incorrectly entered data reduces the designer's efficiency. The designer is encouraged to check data entries before program execution. Another common error is the use of computer programs for conditions beyond the author's intended range of application. Generally, there are conditions which cannot be modelled. It is the designer's responsibility to ensure computer programs are correctly applied.

Calibration and Verification of Hydraulic Computer Modelling

Calibration is the process of varying model input parameters until a good fit between measured and simulated values occur.

- The calibration and verification processes require large amounts of data including surveyed cross-sections, recorded water levels and flow rates. The measurement of data necessary to calibrate and verify the model is a time-consuming and expensive undertaking. If the failure of a facility would increase the risk to life or property damage, then data collection for calibration should be considered.
- The calibration and verification procedure involves varying input parameters until a good agreement exists between measured and simulated values. The following parameters are typical of those varied during calibration:

    - channel and flood plain surface roughness; and
    - flow expansion and contraction coefficients.

- The calibration and verification of all hydraulic models is strongly recommended.

- If the results of calibration and verification are poor, then field conditions and input data should be carefully reviewed before design simulations are carried out.
- A sensitivity analysis of the input parameters should always be conducted. Hydraulic parameters which are varied include roughness coefficients and expansion and contraction coefficients. A parameter which can influence water levels is the Manning roughness coefficient.

Selection of an Appropriate Computational Method

- Most highway design applications simplify flow conditions by assuming steady uniform or steady varied flow conditions.
- Manning's equation should be used with caution as it may not be appropriate for all design applications. It is convenient to use as a quick method for a rough, initial sizing.
- Final design must always utilize the appropriate method based on the applicable flow condition.

Culvert Hydraulics

Typically, in design applications, culvert hydraulics is used to determine the capacity of culvert. Laboratory tests and field observations have shown that there are two major types of culvert flow:

- flow with inlet control; and
- flow with outlet control.

A brief discussion on the culvert hydraulics is provided below. For a detailed discussion on background theory and equations refer to Chapter 8. For design applications, refer to Chapter 5.

Flow with Inlet Control

- Inlet control means that the discharge capacity of a culvert is controlled at the culvert entrance by the depth of headwater and the entrance geometry, including the barrel shape, cross-sectional area and the type of inlet edge.
- The roughness and length of the culvert barrel and the outlet conditions are not factors in determining the culvert capacity.
- The longitudinal slope reduces headwater only to a small degree, and can normally be neglected for conventional culverts flowing in inlet control.

Table 3B.1: Hydraulic Computer Model Applications

Applications	Computer Program									
	MOBED	HYCHAN	FERNS	FLOW 1-D	HEC-2 [1]	HEC-6	HEC-15	WSPRO	EXTRAN	DWOPER
Flow Conditions:										
Steady	•	•	•	•	•	•	•	•	•	•
Unsteady	•	-	•	•	-	-	-	-	•	•
Gradually Varied	•	-	•	•	•	•	-	•	•	•
Rapidly Varied	-	-	-	-	-	-	-	-	-	-
Subcritical	•	•	•	•	•	•	•	•	•	•
Supercritical	•	•	•	•	•	-	•	•	•	•
Two Dimensional	-	-	-	-	-	-	-	-	-	-
Tractive Force	-	-	-	-	-	•	•	-	-	-
Energy	•	•	•	•	•	•	-	•	•	•
Momentum	-	-	-	-	-	-	-	-	-	-
Output:										•
Water Surface Profile	•	•	•	•	•	•	-	•	•	-
Velocity Profile	•	-	-	•	•	-	•	•	-	-
Ice	-	-	-	-	•	-	-	-	-	-
Cross Section	-	-	-	-	•	-	-	•	-	-
Flow Distribution										
Options:										
Tributary Profile	-	-	•	•	•	•	-	•	•	•
Multiple Profile	-	-	•	•	•	-	-	•	-	-
Automatic Calibration	-	-	-	-	•	-	-	•	-	•
Bridge/Culverts	-	-	•	-	•	-	-	•	-	•

Note: [1] HEC-RAS has been issued as an update to HEC-2.

Flow With Outlet Control

- Outlet control means that the discharge capacity of a culvert is controlled by the depth of tailwater including the velocity head within the barrel, entrance losses and friction losses.
- The roughness, length of the culvert barrel, and slope are factors in determining the culvert capacity.

Analysis Approach

- In most cases the operating flow condition of the culvert, is not known.
- This unknown is avoided by computing the headwater depth (refer to Chapter 5 for details) for both the inlet and outlet controls; the higher value then indicates the type of control, and should be used as the governing depth in design.
- This method is relatively accurate except for the few cases where the headwater is approximately the same for both types of control.
- Computational procedures are simplified with the use of design aids (i.e. nomographs-refer to the Design Charts) and computer programs (see Table 3B.1).

Reservoir Sizing and Routing

Typically, in highway design applications, reservoir sizing is used to determine the size and configuration of a stormwater detention facility. Reservoir routing is utilized to determine the effect of a reservoir on hydrograph shape, timing and peak flow. A brief discussion is provided below. For a detailed discussion on background theory refer to Chapter 8. For design applications, refer to Chapter 4.

Reservoir Sizing

- Reservoir sizing is comprised of two components:

  - the physical configuration of the detention facility (volume and shape); and
  - the hydraulics of the outlet structure.

- The physical configuration is quantified by determining volume of storage for various stages of depth or elevation. This relationship is commonly referred to as a stage vs storage representation.
- The hydraulics of the outlet structure is determined by applying basic culvert hydraulics and flow control principles (i.e. orifice controls and weir flow), to determine flow rates at the various stages of depth or elevation. This relationship is commonly referred to as a stage vs discharge representation.

Reservoir Routing

- The operation of any reservoir or stormwater detention facility, and its effect on the inflow hydrograph, is determined by inputting the reservoir configuration and hydraulic representation into a computer program.
- The computer program will route the inflow hydrograph through the facility and intrinsically determine the effects that the configuration and hydraulic representation will have on the hydrograph.
- In the design of stormwater management facilities, routing and sizing are simultaneously completed and the overall design process becomes iterative.
- Reservoir routing is also used in hydrologic modelling.

Stream Channel Sizing and Routing

In highway design applications, stream channel sizing is used to determine the size and configuration of proposed modifications to an existing watercourse. Stream channel routing is utilized to determine effects on hydrograph shape, timing and peak flows. A brief discussion is provided below. For a detailed discussion on background theory refer to Chapter 8. For design applications, refer to Chapter 5.

Stream Channel Sizing

- The physical configuration of a stream channel is quantified by determining volume of storage for various stages of depth or elevation. This relationship is commonly referred to as a stage vs storage representation.

Stream Channel Routing

- The operation of any reservoir, and its effect on the inflow hydrograph, is determining by inputting the physical configuration into to a computer program (see Table 3B.1).
- The computer program will route the inflow hydrograph through the stream channel and intrinsically determine the effects that the configuration and hydraulic representation will have on the hydrograph.
- In the design of stream channel systems, routing and sizing are simultaneously completed and the overall design process becomes iterative.
- Stream channel routing is also used in hydrologic modelling.

Stream Channel Stability Analysis

Channel stability analysis is critical in any stream channel, bridge or culvert design application and is used to determine suitable protection against erosion. For design applications, refer to Chapter 5.

- **Tractive force** analysis is used in the design and analysis of stream channel lining material. Determines shear force that is exerted by the mass of flowing water on the stream channel. Lining material can then be checked to ensure resistance against the shear or tractive force of the moving water.
- **The permissible velocity** method is used in the design and analysis of stream channel lining material. Given velocity in a stream channel, lining material can be checked to ensure resistance against the velocity of the moving water.
- **Scour analysis** is typically applied to piers and bridge abutments and at culvert outlet locations.

Appendix 3C: Evaluation

Drainage is usually only one of many concerns in highway projects. This appendix focuses on evaluation requirements for drainage work alone. Evaluations that encompass a broader range of impacts is beyond the scope of this appendix.

Evaluation is undertaken before decisions are made in order to help in the decision making. It is not a substitute for the decision making process. This appendix focuses on the evaluation task and not on decision making. Moreover, it does not consider budgeting, capital finance, cost control and cost recovery since these are not normally part of evaluation.

What Is Evaluation?

Evaluation is a deliberate, explicit process to help identify the best option when several are being considered. The best option is one that:

- achieves the same results as other options but at a lower cost; or
- costs the same but has fewer adverse effects; or
- costs more but has additional benefits that justify the extra cost.

Do an evaluation when there is a decision to make, for example, consider the following.

- Which option should be pursued?
- Is this a "go" or a "no go" situation?

Evaluation serves no purpose if there is no choice to be made.

Why Do Evaluations?

When a decision is made without an evaluation, the chances of making a poor decision increase. What's a poor decision? One that fails to take all of the costs and benefits into account. For example a poor decision may:

- not be clear about the objectives; or
- ignore future operating and maintenance costs; or
- overlook significant impacts (flooding, fisheries, erosion,...); or
- fail to weigh the concerns of people who are affected by the project.

Input from Earlier Tasks

The evaluation process uses the following types of information from the tasks that precede it:

- descriptions of the impacted social and natural environments;
- descriptions of objectives and criteria; and
- descriptions of highway alternatives and drainage options proposed and the effects of each alternative/option (e.g. achievement of objectives, costs).

What is Included in an Evaluation Process?

There are three basic steps in evaluation:

1) describe the options;
2) determine the costs and benefits for each option; and
3) compare options based on costs and benefits.

What are costs and benefits? Other words for "costs and benefits" are "pros and cons" or "advantages and disadvantages" or "beneficial and adverse impacts." These are not just the monetary costs and benefits. Non-monetary costs and benefits that are measured in quantitative or qualitative ways are included.

Describing the options is completed prior to the evaluation. Descriptions should include:

- capital, operating and maintenance costs; and
- measures indicating the achievement of study objectives (including the avoidance of adverse impacts).

A variety of techniques are used to determine costs and benefits, and to compare options based on those costs and benefits. Impact matrices are one of the most common techniques. Other tools include cost-benefit and cost-effectiveness analysis, and non-monetary rating and ranking methods. Valuation methodologies are discussed in the next section.

A good evaluation process is as follows.

Inclusive:	all stakeholder are identified and included in the process.
Complete:	all beneficial and adverse effects are taken into account.
Transparent:	impacts are described and evaluated in a way that is readily understood.
Objective:	the evaluation process creates information that is accurate and balanced.
Logical:	objectives and criteria are clearly defined and linked to the evaluation of project effectiveness and project impacts.
Iterative:	the evaluation proceeds in stages. First, options that are infeasible, ineffective or inferior are dropped. The remaining options are screened to select a preferred option.

Keep the Evaluation Simple

For most studies, a basic evaluation will be all that is called for. The minimum requirements of a basic evaluation are:

- estimate life cycle costs for all feasible options;
- estimate the effectiveness of every option in satisfying every objective;
- summarize costs and benefits for all the options in an impact matrix;
- consult all major stakeholder and account for their concerns;
- compare options on the basis of their ability to satisfy study objectives; and
- identify the best option and a provide rationale for its selection.

Plan the analysis carefully to meet the minimum requirements. For instance, if you require estimates of expected annual flood damages, then you must complete the necessary background field investigations and hydrologic modeling analyses. If, as is usually the case, some criteria are not assigned dollar values, then a method is needed to compare monetary and non-monetary criteria data.

A more sophisticated approach to evaluation may be required under the following conditions:

- the study area is a complex watershed with multiple competing objectives;
- water management issues have a high public profile and are surrounded with controversy;
- there are prominent stakeholder groups with conflicting interests; and
- regulatory requirements demand methodological sophistication.

If this is the case then services of an expert in evaluation may be required. The evaluation must be objective and fair and it must appear to be so.

Give All Stakeholder an Opportunity to Be Heard

Public participation methods that do not invite dialogue violate this requirement, as do evaluation methods that rely strictly on experts rather than the people who are affected by proposed alternatives.

Do Not Bias the Evaluation by Your Choice of Methods

A commitment to benefit cost analysis, for example, narrows the analysis to those effects that can be valued monetarily and excludes effects that are “intangible”. On the other hand, a decision not to do any costing would leave economic objectives in limbo.

The Evaluation Team

In smaller studies, evaluation can be completed by the technical analyst(s) under the direct supervision of senior staff. In larger studies, an evaluation team may be used to oversee the work. The primary function of the evaluation team is to oversee the technical work and guide the presentation of results. Ideally, an evaluation team includes:

- persons who will help formulate final recommendations;
- persons with technical expertise in drainage and evaluation;
- persons who do the technical work; and
- persons who are involved with the project from the outset.

Analysts who are responsible for the evaluation will require strong quantitative skills, will have a good understanding of drainage work and should have training in basic evaluation techniques (e.g. engineering economics, environmental planning or applied economics).

Public Consultation

Public consultation gives people and organizations who are outside of the planning and design process an opportunity to participate in the process. Public participants may include local politicians, land owners, residents and businesses in the study area, special interest groups and external agencies.

Possible components of a public participation program:

- information dissemination;
- interviews with key informants;
- surveys or focus groups;
- informal meetings to exchange information and resolve conflicts;
- periodic open house meetings;
- public advisory committee participation; and
- public representation at hearings (e.g. E.A. Board, O.M.B., drainage tribunal).

The role of the public in a consultation program can range from passive providers of information to active participants in the planning process. But whatever their role, the public consultation program should not be a “smoke and mirrors” exercise or a “rubber stamp”, for decisions that are already made. Nor should it be used to manipulate public opinion or mounted simply to satisfy E.A. process requirements (i.e. “going through the motions”). Programs that have these motivations risk alienating the public and producing counter productive results such as delays or failures to secure project approvals.

Many potential benefits can follow from effective public consultation. Early consultations can be used to identify public issues and refine the study objectives. Public inputs can also lead to modifications in other aspects of the planning process, for instance, the study area was enlarged after public consultations on a river crossing design project in Timmins (Leveck and Quirion, 1991). Interactions with the public can also help the study team to identify new study options and refine design concepts. This often happens in the process of resolving conflicts with the public that arise from opposition to proposed options. The end result of successful public consultation efforts is speedier public acceptance of projects (Sultan, 1993).

Evaluation Methods

Basic methods that are commonly applied in evaluations are described in subsequent sections.
More elaborate evaluation methods, such as optimization models that may be used in complex studies, are not discussed. Moreover, this appendix does not deal with a range of socio-economic tools that are used to describe economic and community impacts.

Some of the methods of an economic or financial nature are also applied in budgeting, financial planning, cost control and cost recovery, but these topics are not covered here.

Approaches To Valuing Impacts

Value measures the significance that people attach to things. Values are used to understand and compare the relative merits of options and to help decision makers identify preferred options. In evaluation studies, the value associated with an effect depends both on its magnitude and on the importance assigned to that effect by the stakeholder.

Monetary Measures

Prices measure value for goods and services that are bought and sold in markets. Many of the effects of drainage projects can be valued directly using market prices (e.g. capital costs, flood damages, land values), while others can be valued indirectly with market prices (e.g. travel time, recreation).

Non-Monetary Measures

Many criteria are not easily valued using market prices (e.g. aesthetic impacts, health, safety, ecosystem impacts). Methods are available to assign either monetary or non-monetary values to these. Simple non-monetary methods involving low/medium/high ratings are recommended. If you use a rating scheme like this for a criteria, first define what is meant by the terms low, medium and high with respect to that criteria; for example the terms may be assigned on the basis of the estimated magnitude of the impact or its duration. In developing a rating scale, it is advisable to have a rating value below the “low” rating corresponding to a zero or negligible effect level.

Capital Costs

Capital costs are the investment costs incurred at the outset of a project to acquire land, and construct or install long lived physical assets such as earth embankments, bridges, fish habitat structures and armoured channels. Capital costs will include costs for land acquisition, planning and design, construction, and commissioning.

Double counting occurs when an impact is given too much weight in an evaluation because its value is counted twice.

Capital costs measured as the initial cash requirement to pay for the investment. Financial costs, which include depreciation and amortization costs, are not considered in an evaluation exercise. To include both the investment cost and the annual depreciation and interest costs in the analysis, would amount to double counting.

Capital grants affect the manner in which capital costs are distributed among different funding agencies, but they do not affect the overall cost of a project to society. Since public sector projects should be screened on the basis of total social cost, any grants that may be available to offset capital costs should be ignored in an evaluation.

When assessing study options and preliminary design options capital costs are approximated from design data. Project costing at these stages will not be accurate enough to provide the basis for budgeting and tendering of contracts, but it will provide a reliable means of selecting the preferred option.

Capital Cost Analysis:

- include investment costs, engineering costs land costs and other direct project costs; and
- do not include financing costs, amortization, depreciation, or grants.

Estimating Capital Costs - Unit Costing

Unit costing, the principal method used in estimating capital costs, is done as follows:

1) measure physical quantities of standard project components as rough take offs from project design data;
2) cost project components individually as follows: (unit cost) x (no. of units);
3) estimate total project costs as the sum of individual component costs; and
4) add an allowance for engineering and contingencies.

Costing forms are used to summarize the analysis (see Example 3C.1). In addition to providing a summary of the costing calculations, always identify the reference year for the unit cost data in the table.

Unit costing will only provide an estimate of the construction cost portion of capital costs. Other costs are incurred for elements such as engineering and legal services, land acquisition, the relocation of utilities, taxes and unforeseen contingencies. Land costs are the subject of a separate section below. Unit costing may be feasible for certain of these elements while others are estimated as a lump sum amount or by using factors applied to the estimated construction cost. Thus, engineering and contingencies are estimated as percentage of total construction costs as shown in Example 3C.1.

Unit costing at the study options and preliminary design stages will differ in the level of detail and the accuracy of resulting estimates. Key differences in costing work at the two stages are described in Table 3C.1.

Example 3C.1: Project Costing Table for a Stream Channel Realignment and Protection At A New Crossing (1996 dollars)

Item	Units	Quantity	Unit Cost	Total Cost
Dewatering and stream diversion	lump sum	1	10,000	10,000
Strip and stockpile top soil	lump sum	1	5,000	5,000
Earth excavation	m^3	1,000	10.00	10,000
Supply and install gabion retaining wall	m^3	100	150	15,000
Supply and place rip rap & geotextile	tonne	500	20	10,000
Fish habitat measures	lump sum	1	1,000	1,000
Regrade embankments, restore top soil	lump sum	1	3,000	3,000
Seed and mulch embankments	m^2	500	0.50	250
Replace agricultural tile drainage outlets	each	5	200	1,000
Sub-total				55,300
Engineering, Contract Administration			10%	5,500
Contingencies			10%	5,500
Sub-total				66,300
GST			7%	4,600
Total				70,900

Table 3C.1 - Unit Costing of Study Options and Preliminary Design

	Study Options	Preliminary Design
Application	Drainage options developed at a coarse level, minimum of design information (e.g. length of horizontal alignment, number of river crossings); the analysis may simply involve a high to low cost rating or a ranking of projects based on expected relative cost.	Designs developed to a stage that allows rough quantity take offs of standard materials, design details differentiate the alternatives (e.g. box or corrugated culvert, number of bridge piers, type of erosion control).
Accuracy	order of magnitude, ± 30% to 50%	± 10%
Approach	Unit costing of all elements with use of unit costs for major project components (e.g. total cost per storm water pond or per crossing).	Unit costing of specific materials and contract services (e.g. rip rap, earth excavation); quotations may be secured on major structural components.

Operating and Maintenance Costs

Operating and maintenance (OM) costs are incurred on an ongoing basis once a project is built and is use. OM costs are recurring costs. Maintenance costs that occur annually include items such as garbage removal, grass cutting, plant control, inspections, and minor grade maintenance (e.g. repair of rills and gulleys).

Certain costs for major maintenance work occur less frequently (e.g. reshaping of drainage ditches, rip rap replacement, gabion basket repairs, major gulley repairs, site clean up following severe flooding, painting of structures, etc.). Operating costs will not generally be incurred for highway drainage and water management works since there aren’t usually any systems to be “operated” (active treatment systems for urban runoff are an exception that could require power or material inputs).

Estimating Operating and Maintenance Costs

Operating, maintenance and replacement costs should be estimated for each alternative at both the study option and preliminary design stages. At a study option stage, estimate OM costs as a proportion of the original capital cost, for example:

Annual OM Costs = 0.005 x Capital Cost

Such estimates can draw on the experience and judgement of the design engineer and of regional staff involved in maintenance operations. More detailed unit costing may be feasible for options at the preliminary design stage. Key items of information required in costing OM tasks at this stage are gross labour, material and equipment costs and estimates of staff time and other quantities required for specific tasks.

Example 3C.2: Unit Costing for OM Costs

- Maintenance operations on a roadside drainage ditches may entail annual inspections and cleaning and ditch reshaping every 15 years.
- Unit costs can be determined based on a representative one kilometre stretch of ditch.
- Once calculated, this unit cost is used to estimate ditch maintenance costs for all of the options under consideration.

Item	Units	Unit cost	Total Cost/yr
Annual inspection	one person plus vehicle for .25 hr	$25/person hr, $35/vehicle hr	$15
Annual cleaning	two persons plus vehicle for 1 hr	$25/person hr, $35/vehicle hr	$85
Reshaping every 15 years	lump sum contractor fee	$2000/km	$133
Total Annual Cost/km			$233

Valuation of Land Required for Construction

Estimate the value of lands that must be acquired for a project in order to provide a complete picture of project costs. Current market value is the appropriate measure of land value. This value will be similar to prices obtained in recent real estate transactions, and it should reflect the best
and highest possible use of the lands in question.

Several possible sources of land value data are provided below.

- MTO's regional property office and local real estate agents can provide representative market values for the type of property in question.
- Detailed information on recent property sales can be obtained from the land registry office fee books. For each real estate transaction, these books record the sale date, a description of the property, its location, the names of the buyer and seller, the sale price, and the assessment role number for the property. In using fee book data, screen out any transactions that are not at arms length such as sales between family members which may not be made at market value. Sales data taken from the fee books should be for properties that are similar in quality to the acquisition properties.
- A real estate appraiser can be retained to provide a preliminary appraisal of the lands in question. This is a costly alternative that should only be undertaken in cases where land values have become an important or controversial issue. The preliminary appraisal involves a visual inspection of subject properties, and the assembly of data from secondary sources such as the municipal assessment roles. The appraiser does not make any measurements of the properties in question and does not vouch for the accuracy of the assembled data.

Agricultural lands and undeveloped commercial lands can be valued based on the size of the acquisition area. For these cases, land value is expressed as a value per unit area $'s/hectare).

Urban and rural residential properties can be valued on a per lot basis if the entire lot is to be acquired. Where partial acquisition of developed or undeveloped land parcels is required, professional help may be required to assess the loss of value of the affected parcels; this loss is the appropriate measure of value for the affected lands.

Impacts on Adjacent Lands

Highway drainage and water management projects can at times affect the commercial use and the personal use and enjoyment of adjacent properties. The use and enjoyment of property is impacted in a number of ways:

- easements may limit the uses that are permitted;
- environmental amenities may be enhanced or degraded;
- flooding risks may change;
- agricultural productivity may be degraded by soil disturbance, and by changes to surface and sub-surface drainage; and
- commercial operations may suffer due to temporary or permanent restrictions to property access.

During the study option stage, identify likely impacts related to property. Describe the extent and severity of impacts using measures such as the number of households, persons, businesses and properties affected, the size of the affected area, the nature of activities that are affected and the duration of impacts.

Descriptions of impacts are often complex quantitative measures. Wherever possible, summarize these measures as single values for each option so that they are easy to compare across options. Often you may be able to develop a summary index to do this (see Example 3C.3). Alternatively a simple high/medium/low rating of the expected impact can be used.

Example 3C.3: An Index To Measure Cropland Impacts

- Cropland of varying capability is taken out of production to accommodate alternative storm water management facilities.
- A simple measure of total area lost is misleading since the proportions of class 1, 2 and 3 lands vary across alternatives.
- A weighted sum of lost cropland is calculated where the weights measure the productive capability of each class.

Cropland Classification	Affected Cropland Area (ha)	Productivity Rating	Weighted Cropland Area (ha)
Class 1 Class 2 Class 3	5.5 12.0 4.0	1.0 0.8 0.6	5.5 9.6 2.4
Equivalent Class 1 Cropland Area			17.5

Monetary Assessment of Impacts on Adjacent Agricultural and Commercial Lands

Value impacts on adjacent agricultural and commercial operations by estimating changes in the annual income earning potential associated with the affected property. This requires an analysis of annual operating budgets:

1) Estimate annual gross revenues for the pre-project period and for the period of impact as:

   (product sales volume/yr) x (price)

   For the pre-project period use average sales volumes and prices. For the period of impact adjust these averages to reflect the impact of the project. Both the sales volume and price may change and the change may be temporary or permanent. If several products are produced, do this calculation for each product.
2) Estimate the loss in gross revenues for each year of impact as the difference in pre-project and impact year revenues.
3) Compile information on total operating costs and identify any cost savings that may occur in the impacted operation as a result of lower product sales. The most likely source of savings will be reduced purchases of inputs.
4) Estimate changes in net revenues for each year of impact as:

   (loss of gross annual revenue) **-** (annual cost savings)

   The data for a budgeting analysis may at times be obtained directly from the affected operation. Otherwise, secondary data sources must be used.

For impacts on adjacent croplands, consult a representative of the Ontario Ministry of Agriculture and Food to get crop budgeting aids, annual statistical reports and crop insurance records. The crop budgeting exercise will produce a base case or pre-project estimate of the net income per hectare, for the crop or crop rotation found in the study area. A simple and conservative assessment of damages to crop land assumes that all net cropping income from the damaged area is lost for a period of several years while soils recover. A more accurate estimate requires detailed assumptions about yield reductions, rehabilitation expenses, and the duration of the recovery period. The more detailed analysis requires the services of an agronomist.

Data sources for the analysis of criteria related to commercial property impacts are not as complete as those for agriculture (the annual Statistics Canada publication, *Market Research Handbook*, Catalogue 63-224, provides small business financial statistics). The analysis of business income loss is usually left to an appraiser.

Flood Damages

The initial analysis will involve mapping, site inspections to inventory the flood plain structures, and an examination of historical flow and flooding records. It may be possible to guesstimate total expected flood damages by extrapolating from earlier estimates for the study area. If it becomes clear that flooding is a significant risk that can be reduced, then average annual flood damages should be estimated for the base case and for each flood reduction option. The impact of each option is measured as the expected change in damages:

(avg. annual damages after project) - (avg. annual damages before project)

Methods to estimate average annual flood damages are fully documented in the report, *Flood Damages: Volume 2 - Guidelines for Estimation*, (Paragon Engineering, 1984; available from the Ontario Ministry of Natural Resources).

Due to the extensive data required for full flood damage analysis, a non-monetary estimate of flooding potential is more appropriate at the study options stage. This can be measured as a count of the households and structures, and the area of lands classified by land use that are at risk in the flood plain.

Example 3C.4: Measuring the Exposure to Flooding

- The following example illustrates a flood exposure measure that takes probability of flooding into account.
- This measure does not account for the severity of flooding since the degree of damage to each home is not estimated.

Flood Elevation (meters)	Probability of Occurrence of Flood Flows	# Homes Exposed at Each Flow Level	# Of Homes Multiplied by Probability of Occurrence
1005.0	0.100	1	0.100
1005.5	0.050	12	0.600
1006.0	0.010	55	0.550
1006.5	0.001	167	0.167
Annual Average Number of Homes Flooded			1.417

Other Impacts Related To Adjacent Property

Other impacts on adjacent property include changes to the amenities provided by the property and its surroundings, and changes in the uses that can be made of properties. The changes may be beneficial or adverse. Beneficial effects will arise primarily as a result of flood reduction and erosion control. For example, the replacement of an undersized culvert with an adequate culvert at a stream crossing could lower the regional flood line above the culvert. This may ease restrictions on cut and fill activities or allow an expansion of gardening into new areas. Measures to control stream bank erosion can protect riparian property by preventing the loss of land to bank slumping.

Identify and describe the expected beneficial and adverse effects on property uses and amenities at the study options stage and rate their significance as low, medium or high. If a monetary evaluation is also required, then the simplest approach involves assigning a fair market price to the lost amenities. This approach can be used in cases such as the loss of horticultural plants, since these can be valued based on the price of equivalent plants installed by a commercial nursery. In certain cases, the regional MTO property staff have standard values that they apply for purposes of valuation.

At times the lost amenity is not replaceable. In such cases, value the losses by assessing how the property value would be affected by the loss of the amenity. Appraisers undertake the comparative analysis of property values that is required to assign value to these types of impact.

Travel Costs

Travel costs will typically be a subject of concern for a parent highway planning project rather than the drainage and water management component of that project. However, travel costs or cost savings can be significant in a water management context if the options being considered involve longer routes. Longer routes may be required to avoid sensitive water resource features such as wetlands or cold water streams. Travel costs include vehicle operating costs and the value of time spent travelling. Vehicle costs will include only variable operating costs (fuel, depreciation, etc.).

Example 3C.5: Measuring the Value of Travel Time

- This example considers the cost to road users of a 1.0 kilometre diversion around a wetland. In this example the following assumptions are made.

  - The value of time in transit is estimated for all adults in a vehicle and their time is usually valued at 1/3 to 1/2 of their gross hourly earnings (Wage and income information are available from Statistics Canada).
  - The hourly earning figure to use in the travel cost calculation is provincial average hourly income from employment for men and women. Hourly income data for

industry are provided in the Statistics Canada Catalogue 72-002, *Employment, Earnings and Hours*, and annual income data are provided in the Catalogue 72-002, *Income Distributions by Size in Canada*. Divide annual income data by 2000 hours/year to convert it to hourly data.

- The vehicle cost portion of a travel cost calculation is based on unit travel costs. Assuming a unit variable operating cost of $0.25/km, the annual cost on a highway with an annual average daily use rate of 1000 vehicles is:

  Vehicle cost = ($0.25/vehicle.km) x (1 km) x (1000 vehicles/d) x (365 d/yr)
  = $91,250/yr

- The travel time portion of the calculation for this example, assuming an average of 1.2 adults per car and an average hourly wage cost of $15, is as follows:

  Value of travel time = (1/3 x $15/hr) x (1.2 persons/vehicle) x (1000 vehicles/d) x (365 d/yr) x (1 km)/(100 km/hr)
  = $21,900/yr

- The total annual travel cost for this example is:

  Total annual cost = $91,250 + $21,900 = $113,150

Impacts on Recreation

Recreation can be affected if the road work at a crossing creates (or destroys) sport fish habitat or if it makes the stream more accessible to people. Identify and describe such impacts using information from recreational user surveys, site inspections, key informant interviews, attendance records (in the case of managed park facilities) or recreation capacity calculations. Recreation activity is measured in user days of activity. In smaller studies, use a non-monetary valuation for recreation criteria. After recreation resources and activities have been described, assign a low, medium or high rating to the potential impact of each option.

The monetary valuation of recreation criteria resembles unit costing except that the measures of recreational value are called user day values and not unit costs. User day values measure the value of the recreation activity to the user. The value of changes in user days is estimated as:

(number of user days) x (user day value)

User day values range from $10 to $30 for the more common outdoor recreation activities. These values are determined in socio-economic questionnaires.

Intangible values: Recreation is one of many use and non-use values that are called intangible because they are not priced in the market place. Others are health, safety and environmental quality. Questionnaires are used by economists to estimate intangible values.

The analysis of recreation criteria requires a background in economics or geography.

Equivalent Measures of Monetary Value

The various dollar measures of costs and benefits for each option must be added up before options are compared. Before doing this make sure that the estimated dollar values are measuring “apples and apples” rather than “apples and oranges.” Dollar values will be commensurate when they:

- are made over comparable life spans;
- include full life cycle costs;
- are based on the same inflationary price level; and
- account for the time value of money.

Common Life Spans

Project life spans will vary. For example, concrete structures and earth embankments may last 50 years or more while ditches may require major reconstruction every 10 to 20 years. A valid comparison can only be made between alternative projects when life spans are standardized to a common period. This period, commonly set at 50 years for water related projects, represents the minimum expected life span of long lived structural assets. It is also long enough that costs beyond the end point of the 50 year period have little effect on total values, once the time value of money is taken into account.

Assume that projects that will not last for 50 years are periodically reconstructed over the planning horizon so that costs and benefits for all of the alternatives have the same duration.

Life Cycle Costing

One option may have high up front capital costs and low OM costs while another may have low capital costs and high OM costs. A comparison based just on capital costs in this instance would be misleading since high OM costs over the life of one option are ignored. Use life cycle costing to avoid this problem.

With life cycle costing, all recurring costs are forecast over the full planning horizon for every alternative. Life cycle costs include both up front capital costs and subsequent recurring costs. Recurring costs can include annual OM costs, and periodic repairs and replacements.

Accounting for Inflation

Costs are calculated using historical cost data. The historical data will vary systematically through time due to price inflation. Historical cost data must therefore be updated to a common year before calculating unit costs. Where unit costs are obtained from published sources, it is necessary to assure that they are all expressed in terms of the price levels of a common reference year. As a matter of convenience, the reference year for price levels is usually the year in which the costing analysis is done. Inflationary adjustments of historical values are made using an index of price levels. Price indices measure relative price movements over time. The calculation to update values to the reference year is:

$$\text{Price}_{\text{ref. year}} = (\text{Price}_{\text{historical year}}) \text{ x } [(\text{Price index}_{\text{ref. Year}}) \div (\text{Price index}_{\text{historical year}})] \qquad 3.1$$

The reference year for price levels is the most recent year for which price data and price indices are available. This is not the same as the base year or first year of the planning horizon for the study. The base year is usually a year in the near future following the planning period. No attempt should be made to inflate costs forward in time to the base year since reliable forecasts of inflation are difficult to make.

It is not necessary to inflate project costs or benefits over the duration of the planning horizon since a general inflation factor will affect all costs and benefits equally. It therefore has no effect on relative costs and thus on the outcome of the evaluation.

The estimating office of MTO routinely estimates construction cost price indices. Consumer price and construction cost indices are also available from Statistics Canada. Trade journals, such as the *Engineering News Record* and *Chemical Engineering*, publish construction cost indices as well, but these describe American prices and should not be used in Canada.

The Current Value of Future Dollars

Costs and benefits occurring in different years should not be directly added because there is value associated with the passage of time. Future costs and benefits are not given as much weight as costs and benefits occurring today. The interest rate, expressed as an annual percentage, measures the time value of money.

Future dollar values can be added once they have been converted to equivalent present day values or present values. The procedure used to estimate present values is called *discounting*. The discounting calculation is:

$$PV = FV_n \div (1 + i)^n \qquad 3.2$$

where: PV = Present value
Fv_n = Future value in the nth year (n = 1, 2, 3, ..., N)
i = % annual interest rate ÷ 100

For a series of equal annual future values, F, extending from year 1 to year N, the present value calculation is:

$$PV = [F \div (1 + i)^1] + [F \div (1 + i)^2]+ [F \div (1 + i)^3] + ... + [F \div (1 + i)^N] \quad 3.3$$

This expression can be simplified to:

$$PV = F \times [(1 + i)^N - 1] \div [i \times (1 + i)^N] \quad 3.4$$

By convention, the present value in discounting represents value at the beginning of the planning horizon (i.e. on day 1 of year 1) while future values are assumed to all occur at the end of each
year (i.e. day 365 of years 1, 2, 3, ... n).

The interest rate for project evaluation is an annual rate. It is referred to as the discount rate. The selection of a discount rate is based on commercial interest rates as well as rates of return on private sector investments. The selection focuses on long term trends in rates rather than year to year fluctuations and it ignores the influence of inflation on the interest rates.

The discount rate: 7% should be used for project evaluation. Rates of 5% and 9% are used to determine if the outcome of the evaluation is sensitive to the discount rate.

Example 3C.6: Discount Rate Calculations

Initial capital cost in year one		= \$17,000,000
Annual maintenance		= \$100,000
Refurbishment in 30 years		= \$5,000,000
Discount rate		= 7%
Planning horizon		50 years
PV of capital cost	= \$17,000,000 ÷ $(1.07)^1$	= \$15,888,000
PV of maintenance cost	= \$100,000 x $[((1.07)^{50} -1) \div (0.07 \times (1.07)^{50})]$	= \$1,380,000
PV of refurbishment cost	= \$5,000,000 ÷ $(1.07)^{30}$	= \$657,000
Total PV of costs	= \$15,888,000 + \$1,380,000 + \$657,000	= \$17,925,000

Comparing Options

Cost Benefit Analysis

Cost benefit analysis (CBA) is used to compare options based on the present value of their estimated costs and benefits. Present and future costs and benefits for every option are discounted back to the beginning of the planning horizon (the base year), and summed to calculate their present value. The present value of costs is subtracted from the present value of benefits to calculate the *net present value* (NPV) for each option:

NPV of Benefits = (PV of Benefits) - (PV of Costs) 3.5

Net present values can be positive or negative. A negative NPV for an option means that the option can not be justified on the basis of its economic performance. For options with positive NPV's, the one with the highest NPV is the preferred option.

The *benefit-cost ratio,* an alternative summary measure in benefit cost analysis, is simply the ratio of costs to benefits where both are measured as net present values. It can be difficult to interpret and can be an unreliable measure. Do not use it.

Cost Effectiveness Analysis

It is rarely possible to develop dollar measures of all costs and benefits. Frequently it is possible to value only the direct project costs and certain benefits related to cost savings or other readily valued effects. In this situation use cost effectiveness analysis (CEA) to compare options.

For CEA, calculate the net present value of costs:

NPV of Costs = (PV of Costs) - (PV of Monetary Benefits) 3.6

In a simple application, all of the options are equally effective in achieving non-monetary objectives and the option with the lowest NPV of costs is the preferred option. For this case, options which do not meet all of the objectives, including the do-nothing option, are eliminated before hand in this analysis. But life is rarely so simple and a straight forward CEA is not usually possible. In most planning studies, the NPV of costs becomes one of the entries in an impact matrix.

The Impact Matrix

Summarize the criteria data developed in valuing the effects of study options in a table. The resulting "impact matrix" has as many data items as the number of criteria multiplied by the number of options. Dollar values in the impact matrix are reported as present values and costs and benefits are listed separately. Costs are shown as negative values while benefits and cost savings are positive. Depict non-monetary values in the impact matrix table as high/medium/low ratings and describe the associated rating scales in notes to the table.

Cost Benefit Analysis and Cost Effectiveness Analysis alone usually give an incomplete picture of project impact. Combine the NPV of monetary costs and benefits with non-monetary impact data in the impact matrix.

Check the first drafts of the impact matrix to determine whether there are any redundant criteria. A criteria is redundant if all of the options have the same values for that criteria. They all have the same effectiveness or impact with respect to the associated objective if this is the case. Redundant criteria should be dropped from the analysis because they do not help decision makers to differentiate the options.

Example 3C.7 - Sample Impact Matrix

Options	A	B	C	D	E
Present Value of Costs & Benefits (Millions of 1996 $'s)					
Capital costs	($35.0)	($12.3)	($7.0)	($5.5)	($0.0)
OM costs	($1.1)	($0.9)	($1.7)	($0.5)	($4.1)
OM cost savings	$2.1	$0.0	$0.0	$0.0	$0.0
Flood reduction benefits	$8.7	$2.8	$2.7	$1.1	$0.0
Net present value	($25.3)	($10.4)	($6.0)	($4.9)	($4.1)
Non-monetary Criteria Scores					
Wetland protection	medium	low	low	none	high
Protection of fish passage	high	medium	medium	medium	high

Notes on Ratings:

Wetland protection	none low med high	- efforts are made to protect adjacent wetlands - 20% to 50% of wetlands are avoided with a good buffer - 50% to 75% of wetlands are avoided with a good buffer - all wetlands are avoided with a good buffer
Protection of fish passage	none low med high	- no efforts to protect fish passage - fish passage under spring flow conditions - fish passage under average annual flow conditions - no impact on existing fish passage

Using the Impact Matrix in Making Decision

Information in the impact matrix is used to support the deliberations of people who make decisions. While a variety of algebraic techniques can be applied to impact matrix data in order to identify a preferred option, these are not generally necessary. But it is useful at the outset to examine the data to spot any inferior options or a dominant option.

An option is inferior if another option exists that scores as high on every criteria and higher on at least one criteria. Since the second option is equal or superior to the first in every respect, the first can be dropped.

An option is dominant if it scores as high on every criteria as the maximum score reported for all of the other options. That option can be immediately identified as the preferred option because it is superior to all the others.

Example 3C.8 - Inferior and Dominant Options in an Impact Matrix

PROJECTS	A	B	C (inferior)	D	E (dominant)
Present value of project costs ($ million's)	($12.3)	($10.5)	($15.9)	($12.5)	($12.7)
Present value of flood reduction benefits ($ million's)	$0.5	$0.7	$0.7	$2.7	$3.0
Net Present Value ($ million's)	($11.8)	($9.8)	($15.2)	($9.8)	($9.7)
Wetland protection	low	low	low	medium	medium
Protection of fish passage	medium	medium	medium	high	high

Anyone using the impact matrix should have access to a definition of all of the criteria, a description of the options, an explanation of rating systems for non-monetary criteria and supporting documentation for the calculation of criteria data in the matrix. With this documentation, the impact matrix serves as a window onto the evaluation exercise even though it should also stand alone as a summary document.

Decision making is not a trivial task when there is no clear winner among the final options described in the impact matrix. This is the case when the choice between options requires critical trade-offs, for example:

- an option may provide a more complete achievement of some or all of the non-monetary objectives but at greater cost; and
- a greater achievement of one objective can be had only at the expense of another objective (e.g. more flood protection if fish habitat can be removed by channelization).

When facing such trade-offs, the analyst can assist decision makers in several ways:

- plot the impact data to illustrate trade-offs among options;
- review stakeholder information to clarify the significance of each objective;
- conduct sensitivity analysis to determine the stability of the criteria data;
- reassess options and look for compromise options that achieve an acceptable trade-off; and
- reassess the objectives, have factors been overlooked that may help clarify the choice.

Output from the Evaluation Task

The outputs from the evaluation task are:

- summary recommendations concerning the preferred option and its implementation;
- documentation of the evaluation process describing goals and objectives, evaluation criteria, the options, their performance relative to each criteria, and the rationale for dropping options and selecting the preferred option;
- documentation of special technical studies for evaluation; and
- documentation of the public consultation process describing the overall process, listing the time and purpose of meetings and other activities, and reporting feedback from the public including comments about the options (correspondence, petitions, etc.).

Use of the Output

Outputs from the evaluation task are used to inform decision makers about the nature of the planning problem and to guide them in making decisions. Beyond this, the outputs provide inputs to detailed site planning work, providing information and concepts that help to guide engineering design tasks. Evaluation outputs may also be used in the development of an implementation plan including the assignment of responsibilities, project scheduling, budgeting and finance. The estimation and valuation of project effects provides a basis for follow up monitoring and evaluation of the performance of water management measures that are implemented.

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Drainage Management Manual
Part 2

Drainage and Hydrology Section
Transportation Engineering Branch
Quality and Standards Division

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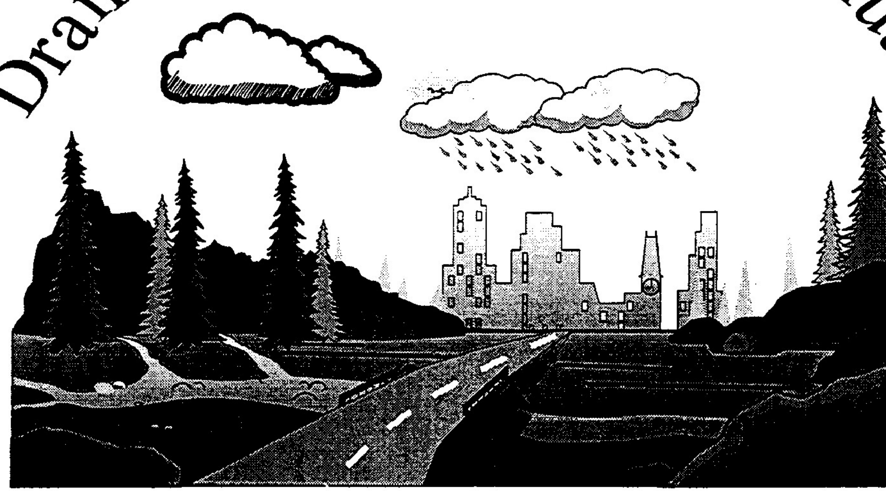

Chapter 4
Surface Drainage Systems

Drainage and Hydrology Section
Transportation Engineering Branch
Quality and Standards Division

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Purpose of This Chapter

The purpose of this chapter is to provide the process and methods for detailed hydrologic and hydraulic analysis and design of surface drainage systems for highway projects such as highway improvement/widening and development of new interchanges and carpools. The components of a surface drainage system discussed in this chapter are:

- roadside ditches;
- storm sewers;
- pavement and bridge deck drainage;
- wet ponds/extended dry ponds; and
- dry ponds.

A surface drainage system does not necessarily include each of these components. The selection of the components to be included in a surface drainage system depends on the needs of the project.

The subjects of this chapter are organized in the same order as the steps of the generic detail design process as shown on Figure 4.1. Other related subjects such as considerations more suitable to the preliminary design stage can be found in Chapter 3 in Part 1 of this manual; theoretical background in Chapter 8 in Part 3; and design charts in Part 4. Detail design of water crossings (bridges and culverts) and stream channels are covered in Chapter 5 in Part 2.

Surface Drainage System Detail Design Process

A generic process for detail design of surface drainage systems is shown in Figure 4.1. It is generally applicable regardless of which components of the surface drainage system are being considered. Where a specific difference may exist between different types of components in any one of the steps in the process, the difference will be discussed in that step.

Step 1: Obtain Information from Preliminary Design If Applicable

Step 1 is critical in that it helps a designer see whether the project in hand is ready for detail design. The reason for this check is that detail design is only one stage of the entire design process of a project as shown in Figure 4.2 (reproduced from Figure 1.1 in Chapter 1). As seen in Figure 4.2, the preliminary design stage has the role to consider and resolve all major design issues concerning drainage. This role must be accomplished before the detail design stage starts if the design is to be a success. This sequence of preliminary and detail design stages should be maintained even if the two stages are done in one design assignment. For guidance on doing a preliminary design, see Chapter 3 in Part 1 of this manual.

For highway projects where preliminary design is appropriate and has been completed, a preliminary design report is normally available and it is expected to indicate whether drainage issues need to be considered. If drainage issues exist, the report is expected to provide all the hydrologic and water quality design information necessary for detail design. Table 4.1 is a list of actions expected of a preliminary design. If some drainage issues need not to be considered, the report is expected to clearly indicate so.

It is not practical to make Table 4.1 an exhaustive list. Therefore a designer must ascertain whether some issues that are not listed may apply to his or her design task in hand.

If a designer determines that some major drainage issues are not addressed, or not addressed satisfactorily, it is prudent not to proceed further with the design. Instead, completing the preliminary design should be considered. Refer to Chapter 3 to review the considerations appropriate for the preliminary stage of design.

Normally, a certain amount of data will have been collected and used in addressing the major issues in a preliminary design. Table 4.2 is an example of the typical data used. If the data are not available or appear incomplete or questionable in quality, the designer should ascertain whether to seek clarification before proceeding to Step 2, or to collect additional necessary data in Step 4.

Figure 4.1: Surface Drainage System Detail Design Process

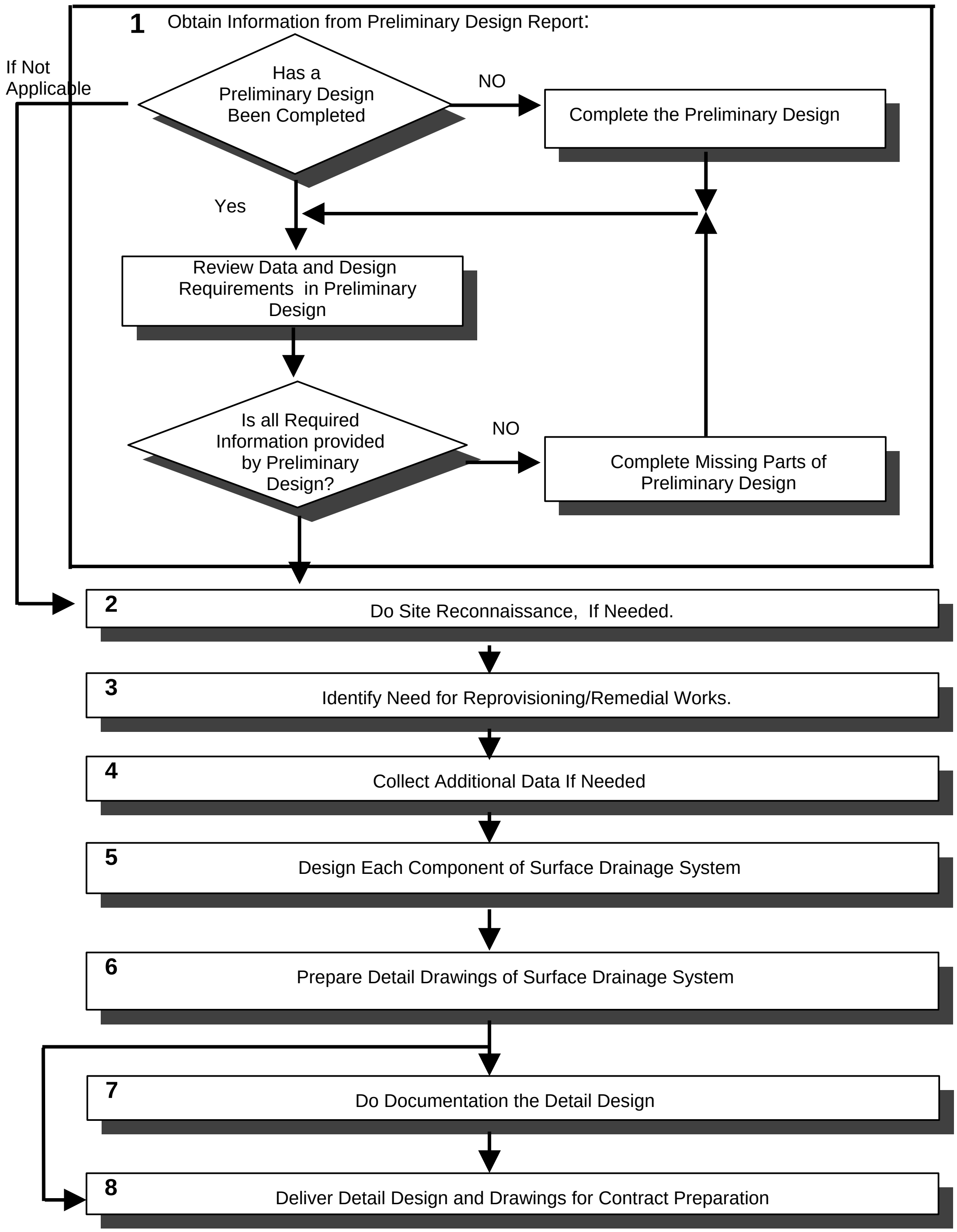


Figure: 4.2 Drainage Management Planning and Design Process

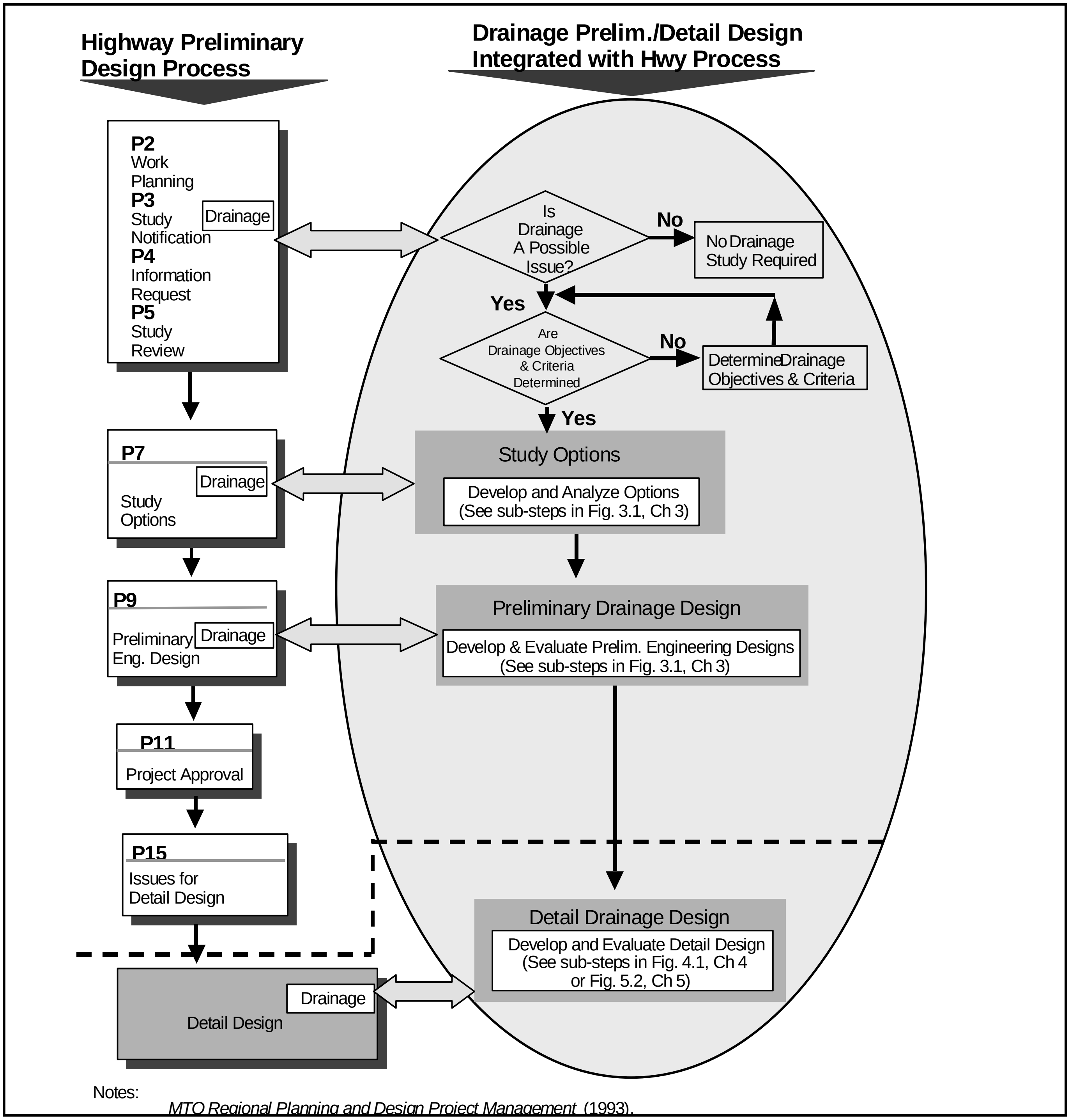


Table 4.1: Expected Actions in Preliminary Design to Address Issues

To Address Hydrologic Impacts

- Watersheds defined.
- Present and future land uses accounted for.
- Overland flow paths for Regulatory and major[1] storms provided for.
- Flood plains maintained.
- Stormwater management for upstream development resolved.
- Drainage inundation impacts on land from highway mitigated.
- Layout of surface drainage system for Regulatory and major storms developed. Key features determined:
    - Overland flow paths.
    - Routes and control water surface and invert elevations of ditches/sewers.
    - Special provision for erosion protection at outlets and embankments (if necessary).
    - Locations of minor flow outlets into major flow ditches/sewers.
- Records of major assumptions made; issues outstanding and follow-up actions required.
- Layout of pond (if proposed) developed. Key features determined:
    - Location.
    - Capacity.
    - Control elevations.
    - Inlet and outlet locations, types and capacities.
    - Emergency bypass location and capacity.
    - Maintenance access.
    - Special safety and maintenance requirements, if any.
- Design criteria determined:
    - Selected Regulatory storm.
    - Frequency and duration of major storm.
    - Allowable water elevation and flow velocity in major flow ditches/sewers.
    - Control tailwater at outlets of major flow ditches/sewers.
    - Stormwater quantity management requirements, if applicable.

To Address Stormwater Quality Impacts

- The same action items as for hydrologic impacts[2].
- Subdrainage areas defined for stormwater quality management.
- Design criteria for stormwater quality management determined.
- Additional features of layout of surface drainage system developed[2]:
    - Arrangement of ditches, sewers, swales and ponds that make up the stormwater quality facility.
    - Location and type of device splitting stormwater quality management flow from storm quantity+quality management flow .
    - If wet pond, capacity and surface elevation of permanent pool.
- Additional design criteria determined[2].
- Other stormwater quality management requirements, if applicable.

Notes:

1. For a discussion of "major" and "minor" drainage, see Chapter 8.
2. To acknowledge the necessity of integrating stormwater quantity and quality management.

Table 4.2: Typical Data Used in Preliminary Design

- Watershed maps:
  - Topography and existing overland flow paths and stream courses.
  - Present and future land uses.
  - Flood plain boundaries.
- Precipitation:
  - Regulatory storm.
  - Frequency and duration of major storm.
	- Time series or statistics of long-term records (stormwater quality management only.)
	- Proposed land allocation for stormwater management facilities (if applicable).
	- Existing surface drainage systems upstream/downstream and on highway r.o.w.

Step 2: Do Site Reconnaissance If Needed

The designers of a project are the best judges to determine whether or not a site reconnaissance is needed. Generally, if any of the following circumstances exists, it may be desirable to undertake a site reconnaissance:

- the designer is not familiar with the site conditions;
- complex grading of the site may be required for some of the drainage works;
- there is a history of drainage complaints (see also Step 3);
- some reprovisioning works may be needed (see also Step 3);
- the proposed drainage system is complex;
- there are possible concerns from neighbouring property owners about the project; and
- follow up on the advice or recommendations of the preliminary design report.

Important observations made from a site reconnaissance should be documented for future actions and reference.

For further discussion of field investigations, see Chapter 7.

Step 3: Identify Needs for Reprovisioning/Remedial Works

Not every project will need reprovisioning/remedial works. The designers should check if a need exists. Examples of possible reprovisioning/remedial works include:

- providing an outlet for an existing ditch that has no outlet;
- adding a maintenance hole to an existing sewer where clogging occurs frequently;
- re-designing the spacing of catchbasins for an existing segment of roadway which experiences frequent ponding;

- re-directing surface runoff from an existing roadway to avoid inundating adjacent property;
- improving the grade of an existing roadside ditch which requires extensive spring ditching;
- adding erosion protection to an existing sewer outlet on an embankment which is severely scoured by the sewer discharge; and
- replacing a defective ditch or sewer outlet for an existing pavement subdrainage system.

Possible sources to help identify needs for reprovisioning/remedial works include:

- Findings from a site reconnaissance (see also Step 2).
- Findings and recommendations of a preliminary design report.
- The nature of the works to be designed. For example, a proposed roadside ditch that crosses the vehicular entrance of a farm may need to be replaced with a sewer underneath the entrance to maintain access to the farm.
- Ministry commitments made previously.
- Reports from regional and district staff.
- Records of complaints from the public or property owners.

If a potential need is identified, determine the merit, nature and scope of the need as well as requirements for design of the reprovisioning/remedial works. This determination may involve additional site reconnaissance, data collection and analysis.

Step 4: Collect Additional Data If Needed

Table 4.3 shows typical data requirements for the detail design of the components of a surface drainage system. Some of the data may have been collected during the preliminary design (see Step 1), but require refinement, such as a site plan at a larger scale.

Table 4.3: Possible Additional Data Requirements

- Site plan of highway project. - Site survey of area for stormwater management facilities. - Frequencies and durations of design minor storms. **Note:** Design criteria for stormwater quantity and/or quality management are normally determined in the preliminary design stage.	- Detail highway geometric design and grading plans. - Soil investigation reports. - Data and design requirements for reprovisioning/remedial works, if applicable.

Designers may have their own sources of data. Nevertheless, Chapter 7 provides a listing of data sources.

Step 5: Design Each Component of the Surface Drainage System

Layout Plan of Surface Drainage System

A layout plan of the surface drainage system for the highway project should be made available by the preliminary design. It should include all ditches and sewers for the Regulatory and major flows and stormwater quantity and/or quality management facilities if applicable. See Step 1. Refine the layout as necessary to suit site conditions. If this refinement may result in changes in design criteria, major assumptions and/or configuration of the surface drainage system of the original preliminary design, the designer should ask for guidance from the project manager or engineer before attempting to make the refinement.

Layout Plan of Subsidiary System

The layout plan of a subsidiary system for minor flow ditches and sewers may not exist at this stage. In such cases, the designer may need to create the layout first, using the following discussion as a general guide.

Roadside Ditches

Roadside ditches, whether they are for major flow, minor flow or both, usually follow the highway horizontal alignment and offset at a given distance from the road shoulder. Very little variation from this general arrangement is expected.

Ditches for minor flow should only discharge to a receiving drain (ditch or sewer) at a designated location(s) determined by the preliminary design.

Storm Sewers

The choice between ditches and sewers is normally determined by the preliminary design. Sewers are seldom used for conveying the major flow. A common use of sewers is for conveying the minor flow from roadway curbs, gutters and catchbasins in urban areas.

Where applicable, a sewer alignment should be located within the designated zone in the highway right-of-way reserved for sewers. Generally, the alignment should be clear of traffic lanes to avoid traffic interruption and to provide maximum safety for workers during sewer maintenance. If this is not possible, the alignment should be confined to one traffic lane and placed in the middle of the lane. Like ditches, a sewer network should discharge to a receiving drain at a designated location(s).

Drainage for Highway Facilities

Within the site of a highway facility, for example a carpool area, the minor flow from the areas used by cars and pedestrians are drained by sewers and catchbasins, with curbs and gutters if desired. The other areas may be drained by ditches, sewers or a combination of both, as local conditions and design considerations permit. The major flow routes should be considered as early as possible in the design process, to determine if water quality/quantity control is required or the flow will be permitted to drain off site without control. The minor flow system should direct the flow to a receiving drain at a designated location(s).

In general, the design considerations are mostly concerned with cost, maintenance, aesthetics and local preference. The preparation of a layout for a subsidiary system is usually carried out under the direction of the project manager or engineer. Use the following discussion as a general guide for preparing a layout.

- Use the proposed grading plan of the site as a base plan.
- Draw boundaries of drainage catchments according to the grading of the site.
- Draw arrows to show the directions of flow of the catchments.
- Select the alignment of the proposed trunk drain such that it will pick up the flow from all the catchments. The outlet of the trunk drain should be at the lowest point of the site. It must be able to drain to a receiving drain at a designated location.
- If no outlet location has been designated, it should be determined in consultation with all parties concerned. Some of the parties may have special design requirements.
- Add the branch drains from the outlying areas, upstream of the trunk drain.
- Oil/grit separators are seldom used with highways. However, there may be a need to consider providing one for a highway facility. This need would normally be determined at the preliminary design stage.

Design Methods

The working details of the design methods are discussed in the sections after Step 8. One section is dedicated to each type of drainage component and the section title is the same as the drainage component being considered, for example roadside ditches. In addition, a summary of the commonly used computation methods is provided in Appendix 4A.

Step 6: Prepare Detail Drawings of the Surface Drainage System

Most detail design drawings and specifications should be defined by a standard, such as the Ontario Provincial Standards Specifications and Drawings (OPSS and OPSD). Prepare a special drawing or specification only if it is absolutely necessary. When such an occasion arises, consult with other designers and offices who may have an interest or concern with the proposed details.

Step 7: Document the Detail Design

The documentation of the detail design will not be separate from the overall highway project documentation process. In general, documenting the results of the detail design facilitates the review by the project manager or engineer and organizes the design information for future reference. Table 4.4 shows what is expected of the detail design documentation of the surface drainage components of the highway project.

Step 8: Deliver Detail Design and Drawings for Contract Preparation

This step is not separate from the overall highway planning and design process. This step wraps up the detail design process.

Table 4.4: Expected Output from Detail Design

Drainage Component	Output of Analysis	Output of Const. Details*
General	- Information from preliminary design, e.g. design criteria.	
Roadside Ditches	- Design criteria. - Methods used. - Major data and assumptions. - Design rainfall. - Design flows. - Ditch geometry. - Ditch lining materials. - Ditch invert elevations. - Calculation results.	- Ditch horizontal and vertical alignments together with highway orientation. - Ditch lining materials and typical cross sections. - Details of inlet placement. - Details of connections with other ditches and sewers.
* Also mention the standard drawings to be included as appropriate.		

Table 4.4: Expected Output from Detail Design (Contd.)

Drainage Component	Output of Analysis	Output of Const. Details*
Storm Sewer Network	- Design criteria. - Methods used. - Major data and assumptions. - Design rainfall. - Design flows. - Pipe materials and sizes. - Invert elevations at maintenance holes. - Elevations of maintenance hole covers. - Calculation results.	- Sewer network alignment plan and vertical profiles together with highway orientation. - Table to show the following details: -ID no., invert elev. and cover elev. of M.H. -Material and size of each pipe segment. -Invert elev. of inlet and outlet of each pipe segment. - Special trenching, bedding and backfilling, if any. - End treatment, if any.
Pavement Drainage	- Design criteria. - Methods used. - Design rainfall. - Curb, gutter, and inlet types. - Calculation results.	- Inlet locations and types shown on sewer network plan.
Extended Dry Ponds, Wet Ponds, Dry Ponds. * Also mention the standard drawings to be included as appropriate.	- Design criteria. - Methods used. - Major data and assumptions. - Dimensioned sketches of plans, typical profiles, cross sections and essential details, e.g. inlet and outlet geometry, emergency bypass, etc. - Calculation results.	- Dimensioned layout plan of facility and its location, connections of facility with incoming and outgoing flows. - Dimensioned typical and special profiles and cross sections and control elevations. - Dimensioned details of typical and special appurtenances, e.g. inlet, outlet, emergency bypass, safety features. - Landscaping (check with landscape designer).

Roadside Ditches*

Detail Design Considerations

At the start of the detail design task, consider the following points where applicable, and review them as the task progresses.

Ditch Cross Section and Lining

For minor flow ditches, try standard grassed ditches first. If they do not have adequate capacity, try a trapezoidal shape with the same side slopes as the standard ditches. Major flow ditches normally need a special cross section that can handle both the minor and major flows satisfactorily. Such a cross section may consist of a triangle or small trapezoid at the bottom and a wider and deeper trapezoid on top of it. See Design Example 4.2. The lower trapezoid will carry the minor flow, and the upper one the major flow.

Major flow ditches may require riprap lining if the flow velocities are too high for grass lining. Gabion is generally not encouraged because of its maintenance problems and poor aesthetic quality. Soil bioengineering may be a feasible alternative. Concrete lining should not be considered except in very rare situations where there is no feasible alternative.

Profile, Invert and Crest Elevations

The ditch profile will normally follow the site topography and the vertical profile of the highway. Ensure that the ditch profile provides the required capacity for the flow along the full length of the ditch to the outlet. The invert should be low enough to intercept all tributary flows, including those from adjacent property and outlets of pavement subdrains. Allow for freeboard between the crest of the ditch banks and the maximum possible water elevation in the ditch. Freeboard is normally 0.3 m or two times the velocity head of the design flow, whichever is the greater. Regardless of this provision, the design water surface elevation in a ditch during minor flows must not be higher than the inverts of the outlets of pavement subdrains.

* This section is a continuation of Step 5 of the Detail Design Process.

Very Flat Terrain

If the terrain is very flat, such that it is difficult to maintain a velocity of about 0.3 m/s at the design flow to minimize sedimentation, consider possible special arrangements such as:

- Divide the ditches into shorter lengths and providing an outlet for each shortened ditch. This helps increase the slope of the ditches without making them too deep. This alternative, however, may not be practical where additional outlets are not readily available.
- Same as above, but allow each ditch to discharge into an intercepting storm sewer. The sewer may need to be deep to provide the necessary gradient. This alternative may not be cheap.
- Provide a pond to temporarily store a portion of the flow from a heavy storm and let the flow discharge gradually into the ditches. This can protect neighbouring areas from flooding. The pond should have an adequate outlet. A decision to provide a pond should normally be made in the preliminary design stage.

Very Steep Terrain

The flow velocity in a steep ditch will normally be high. Therefore, the ditch lining material selected must be able to withstand the shear stress. Minor flow ditches may incorporate drops or check dams to slow down the flow if site conditions and traffic safety conditions permit. Major flow ditches may require energy dissipators and special erosion protection. See Chapter 5. Always check if a hydraulic jump may occur by checking the critical depth or the Froude number of the flow. A blocked steep channel may cause serious erosion of the slopes.

Roadside Safety

Consider using as flat a side slope as possible such as those used in standard ditches. Ditches accessible to people, especially children, should conform to safety requirements. Check dams and weirs can be potential safety hazards to errant vehicles. Consult highway engineers as necessary.

Porous Soil

Normally, a roadside ditch is far enough from a roadway not to cause a significant concern of exfiltration of ditch flow into the road subgrade. But if the soil is very porous and the ditch is close to the roadway, review the situation with geotechnical professionals.

Limited Availability of Right-of-Way

This problem should not arise if the preliminary design of the project was done adequately and the problem was identified and resolved early enough in the design process. If this problem is left to the detail design stage, it may be difficult to find adequate solutions at this stage. Discuss the problem with the project manager or engineer. The solution will depend on the individual situations.

Entrances to Adjacent Property

Where a roadside ditch crosses an entrance to an adjacent property, suitable provision is necessary to maintain access to the property and to maintain uninterrupted flow along the ditch. At the entrance, the ditch is usually replaced by a sewer pipe. This pipe should have the same capacity as the ditch. The ditch and sewer inverts at the entrance should be aligned with each other so that sediment may not accumulate at the connections and ponding in the ditch and sewer avoided. If the crown of this pipe is higher than the design water surface level of the ditch, the cross sectional area of the pipe used for calculating its capacity should be limited to its wetted area only. In any case, the pipe should not be smaller than the minimum size (usually 300 mm dia.) required for maintenance.

Aesthetic Considerations

Aesthetic treatment of a roadside ditch does not necessarily mean expensive amenities. Careful contouring of the ground surface along the ditch and some roadside vegetation will go a long way to beautify the highway. Consult landscaping colleagues if available.

Alternative Design of Details

Alternative designs of details of roadside ditches are limited. Possible considerations are:

- different cross sections of a ditch if non-standards are required;
- different types of grass lining for better adaptation to soil types and moisture conditions; and
- reducing the flow velocities, where required, by increasing ditch roughness or by providing drops or check dams along the ditch.

Design Examples of Roadside Ditches

Three design examples of roadside ditches are provided:

Design Example 4.1: Standard Roadside Ditch for Minor Flow
Design Example 4.2: Non-standard Roadside Ditch for Major Flow
Design Example 4.3: Non-standard Ditch for Conveyance and Sedimentation

Design Example 4.1
Standard Roadside Ditch for Minor Flow

Required

Check whether a grass-lined standard roadside ditch is adequate for the given design conditions.

Given

- Drainage area and road details as shown in Figure (a).
- Standard ditch OPSD 200.02. Grassed-lined. Outlet to nearest watercourse. No tailwater.
- Design for 5-yr and 10-yr storms. Use AES data of Pearson International Airport.

Method 1 (By First Principle)

Method 1 is intended to provide a review of the basic design principle for readers who may be interested.

1. Divide the drainage area into sub-catchments along drainage divide lines. Further subdivide each sub-catchment, if necessary, such that each sub-catchment has a unique land use. This design example has four sub-catchments.

2. Select an applicable hydrology method for estimating design flow. In this example, the Rational method is selected.

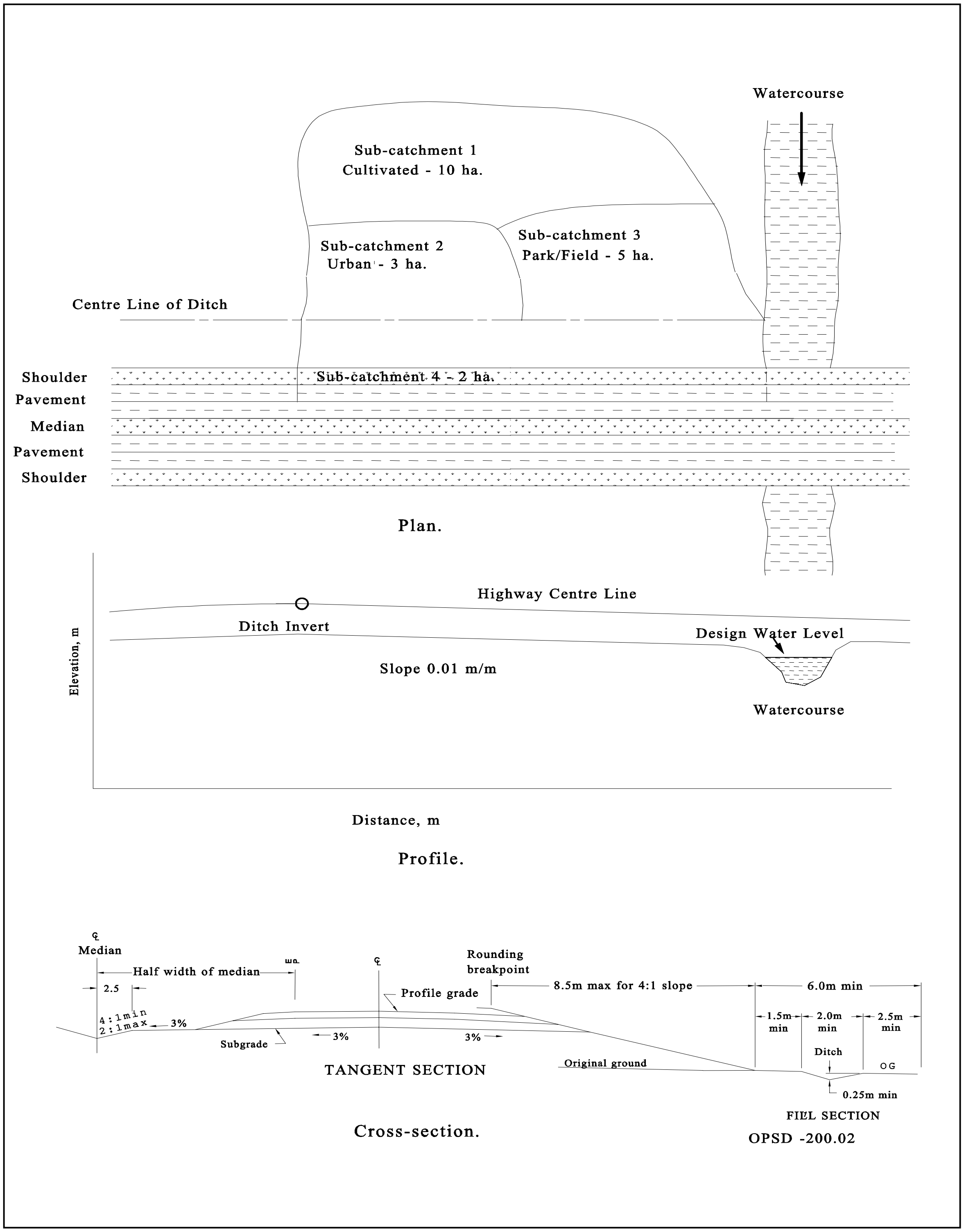
3. Select a suitable raingauge station. For this design example, AES station at Pearson International Airport is used.

4. Calculate the composite runoff coefficient, using Equation 8.10.

5. Calculate time of concentration, T_c. For runoff coefficient C > 0.40, use the Bransby-Williams formula.

$$Tc = \frac{0.057 * L}{S_w^{\ 0.2} * A^{0.1}} = \frac{0.057 * 1025}{0.75^{\ 0.2} * 20^{\ 0.1}} = 45.8 \text{ min} \qquad (8.15)$$

Figure (a): Site of Design Example 4.1



SubCatch.	Area, A (ha)	Land Use	Runoff Coeff. C	A * C
1	10	Cultivated	0.45	4.50
2	3	Urban	0.65	1.95
3	5	Park/Field	0.20	1.00
4	2	Highway/Shoulder	0.90	1.80
Total	20			9.25
			Composite C = 9.25/20 = 0.46	

6. Calculate rainfall intensity from AES data.

Return Period	Coef. A	Coef. B	Intensity, I = A * T ** B (T = Time of Concentration, h)
5-yr	29.8	- 0.711	$29.8 * (45.8/60)^{-0.711} = 36.07$ mm/h
10-yr	34.9	- 0.705	$34.9 * (45.8/60)^{-0.705} = 42.17$ mm/h

Tip: An easier way is to use AES data chart. See Figure (b).

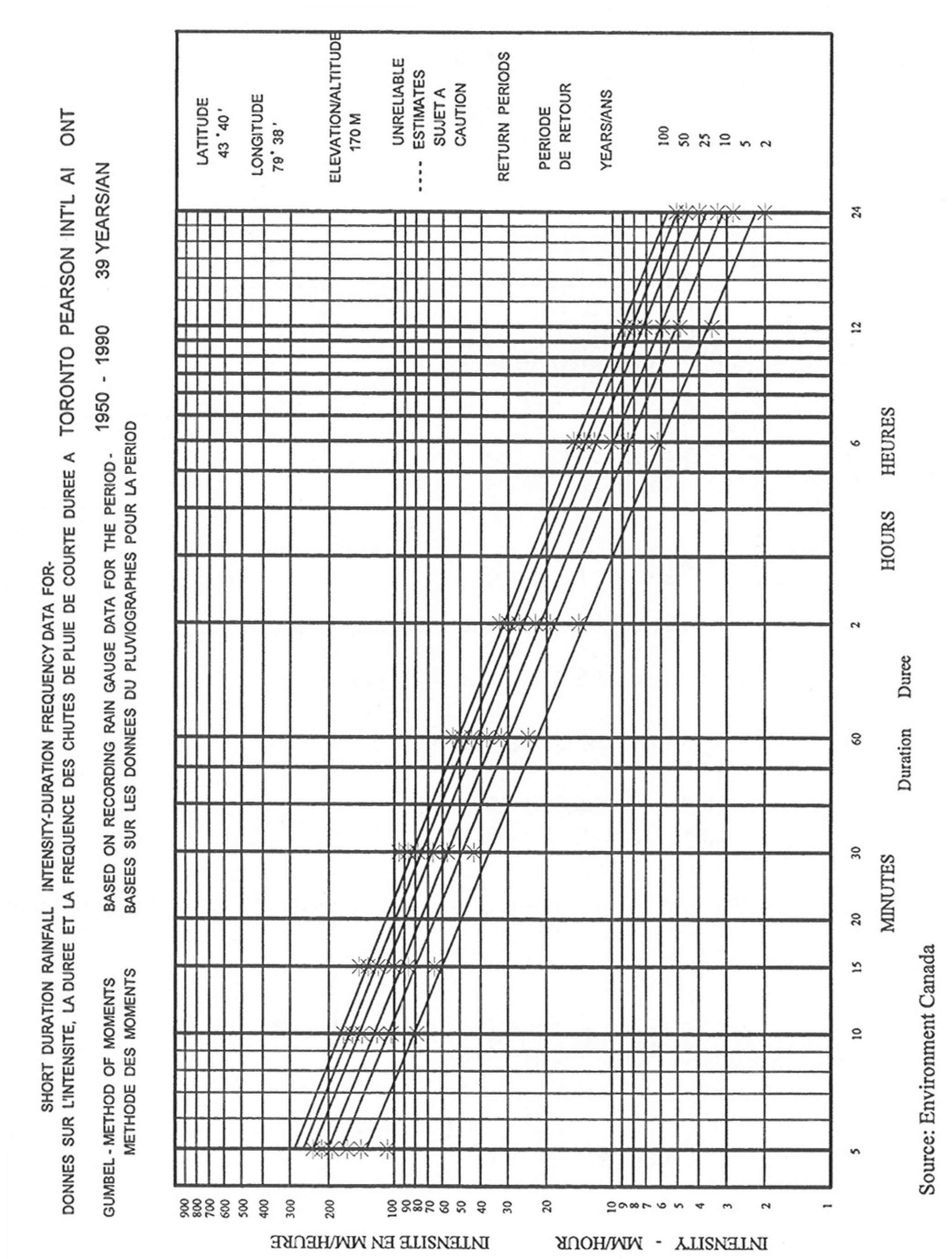
7. Calculate design flow, using Rational method.

$Q_{des5} = 0.0028 * 0.46 * 36.07 * 20 = 0.93\ m^3/s$ (8.19)
$Q_{des10} = 0.0028 * 0.46 * 42.17 * 20 = 1.08\ m^3/s$

8. Calculate the conveyance capacity of the ditch by trial and error, using Manning equation, (8.66).

Input data of ditch:
Grass mixture, erosion resistant soil, slope range 0 ~ 5%
Ditch slope, S = 0.010 m/m
Side slope = 4:1
Manning's n = 0.050 (for grassed channels) (Design Chart 2.01)
Max permissible velocity = 1.5 m/s

Figure (b): AES Intensity-Duration-Frequency Curve



For 5-yr Storm:

Try flow depth	= 0.54 m	
Flow area, A	= 1/2 (2.16 + 2.16) * .54	= 1.1664 m^2
Wetted perimeter, P	= 2 $(2.16^2 + 0.54^2)^{1/2}$	= 4.452 m
Hydr radius, R	= 1.1664 / 4.452	= 0.2619 m
Capacity, Q_5	= $(1/0.05)(1.1664)(0.2619)^{0.667} (0.01)^{0.5}$	= 0.96 m^3/s
Flow velocity, V_5	= 0.96 / 1.1664	= 0.81 m/s

For 10-yr Storm:

Try flow depth	= 0.57 m
Capacity, Q_{10}	= 1.08 m^3/s
Flow velocity, V_{10}	= 0.85 m/s

9. Check possible unstable flow condition, using Froude number formula.

For 5-year Storm:

Top width, T	= 4.32 m		
Hydr depth, y_m	= A / T = 1.1664 / 4.32	= 0.27 m	
Froude No., F_r	= 0.81 / /(9.81 * 0.27)	= 0.49 < 1 (stable)	(8.55)

For 10-year Storm:

Froude No., F_r	= 0.85 / /(9.81 * 0.29)	= 0.51 < 1 (stable)	(8.55)

Method 2 (MTO Rational Drainage Model)

Method 2 calculates the design flows by using the MTO Rational Drainage model. The remainder of the calculation is the same as above and is not repeated.

For working details of the model, refer to the model's user manual. Table (a) shows the input file for the model run and Table (b) the results.

Table (a): Input File

Datafile:C:\Rational\Examp4_1.dat

(Header)

Project No . . WP xxx
Region . . . . Central
District . . . Toronto
Highway. . . . XXX

Comments . . . Example of Data File
IDF Information based on AES Climatic Station.
Time of Concentration based on Watershed Info.

Number of Drainage Areas:	1		
Rainfall Source:	4		Toronto Pearson Air
Number of Return Periods:	2		
Ret Period:	5	10	
Time of Concent Source:	2		

Drainage Area ID	No. of SubLand Use Areas	Land Use Area (ha)	Land Use Runoff Coef.	Catchment Storage Factor	Catchment Length (m)	Catchment Slope (%)	Land Use
1	4	10.00	.45				
		3.00	.65				
		5.00	.20				
		2.00	.90				
		20.00	.46	1.00	1025.00	0.8	Rural

Table (b): Output File

Output File:C:\Rational\Examp4_1.out

(Header)

Project No . . WP xxx
Region . . . . Central
District . . . Toronto
Highway. . . . XXX

Comments . . . Example of Data File

IDF Information based on AES Climatic Station.
Time of Concentration based on Watershed Info.

Number of Drainage Areas:	1		
Rainfall Source:	4		Toronto Pearson Air
Number of Return Periods:	2		
Ret Period:	5	10	
Time of Concent Source:	2		

Output Summary

Drainage Area ID	Drainage Area (ha)	Time of Conc (min)	Runoff Coef.	Return Period (yr)	Rainfall Intensity (mm/h)	Discharge (m^3/s)	Land Use Rural
1	20.00	45.3	.46	5	36.40	0.94	
			.46	10	42.56	1.10	

Summary

Storm Frequency	Design Flow (m^3/s)	Capacity (m^3/s)	Velocity (m/s) Permissible = 1.5	Froude No.
5-yr	0.93	0.96	0.81	0.49
10-yr	1.08	1.08	0.85	0.51

As shown in the above summary, the design capacities for the 5-yr and 10-yr storms are adequate as are the flow velocities. The flows are subcritical.

Design Example 4.2

Required

Design a non-standard roadside ditch with suitable lining for a highway to convey the major flow. Consider both free outfall and submerged outfall conditions.

Given

- The drainage area and road details, as shown in Figure (a).
- The ditch invert grade to follow the roadway grade, 0.50%.
- The ditch discharges to a nearby watercourse.
- Design for the 10-yr storm (minor flow) and Hazel storm (Regulatory flood). For the 10-yr storm, use the AES data of Pearson International Airport.
- The tailwater elevations at the ditch outlet are:
  - 10-yr storm, 101.25 m
  - Hazel storm, 102.25 m

Preliminary Discussion

- The need to provide a roadside ditch for the Regulatory flood is normally examined at the preliminary design stage. This design example assumes that the design criteria are obtained from a preliminary design report.
- A major flow ditch should be designed to accept overland flows from all upstream tributary areas of the watershed under all design storm conditions. Because a roadside ditch follows the road profile, this consideration is normally addressed through suitable planning of the highway profile in which the ditch is located.
- A standard ditch will not have the necessary conveyance capacity for the Regulatory flood, so a non-standard ditch is required. For this design example, and as common practice, a double trapezoidal cross section is used. See Figure (a). The lower trapezoid conveys the 10-yr flow. The major flow will utilize the entire cross section.
- The major flow may create great erosive force on the ditch surface. Therefore, the need for erosion protection lining must be considered.
- A major flow ditch must also be designed to convey the flows of minor storm events without causing operation problems. For example, the flow velocity must not be too low to cause silt to deposit in the ditch.
- Adequate freeboard should be provided for in the design.
- This design example consists of four tasks:

- X Task 1: Determine the design flows, using OTTHYMO (also called INTERHYMO) model. For working details of the model, see the model's user manual.
- X Task 2: Select a suitable ditch cross section, using MTO Open Channel model.
- X Task 3: Do the detailed hydraulic analysis of the selected section, using HEC-2.
- X Task 4: Design the erosion protection measures for the ditch.

Task 1: Determine Design Flows

1. Select a hydrology method. The Rational method is not applicable for two reasons:

   - X The total drainage area is much greater than 100 ha.
   - X Hurricane Hazel is a historic storm which cannot be assigned a frequency. Therefore, the concept of intensity-duration-frequency does not apply.

2. Divide the drainage area into sub-catchments along drainage divide lines. Further subdivide each sub-catchment, if necessary, such that each sub-catchment has a unique land use. See Figure (a).

3. Calculate input parameters from the data of the rural drainage areas:

   (a) Average ground slope, S = 1%
   Length/Width, L/W = 0.75

   (b) Determine CN values and calculate composite CN. (Design Chart 1.09)

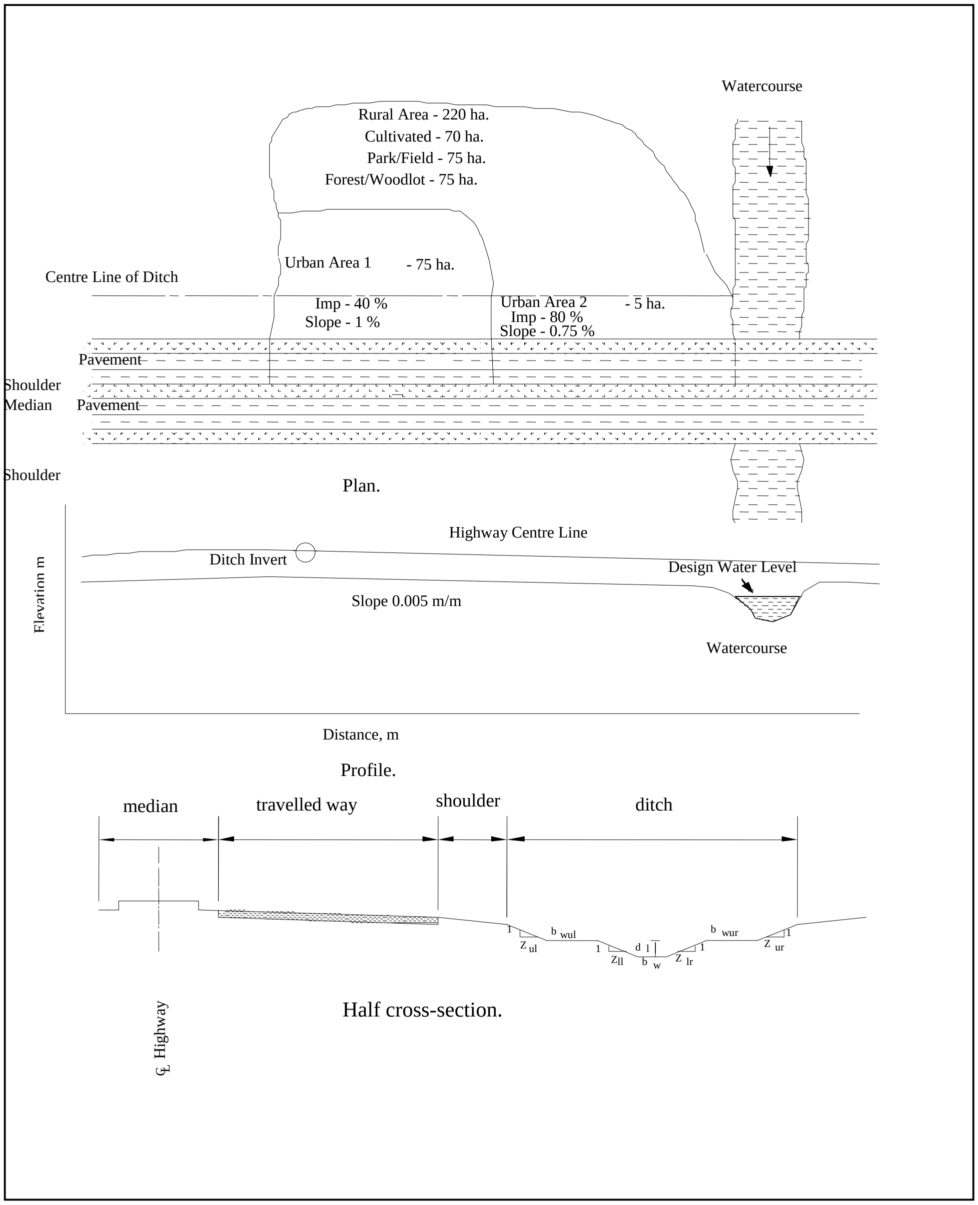
Land Use	Area, A (ha)	CN	A * CN
Cultivated	70	74	5180
Woodlot	75	58	4350
Park/field	75	65	4875
Total	220		14405
		Composite CN = 14405/220 = 65	

   Convert CN for AMC II (= 65) to CN for AMC III for Hazel storm:
   Composite CN = 81 (Design Chart 1.10)

   (c) Estimate the time to peak, T_p, for slope less than 2%.

$$T_p = 0.0086 * A^{0.422} * S^{-0.46} * (L/W)^{0.133} \quad (8.34)$$
$$= 0.0086 * 220^{0.422} * 0.01^{-0.46} * 0.75^{0.133}$$
$$= 0.67 \text{ h} = 40.2 \text{ min}$$

Figure (a): Site of Design Example 4.2



4. Calculate the values of input parameters for the urban areas.

(a) Urban Area 1

AREA	= 73 ha
Ratio of impervious area, TIMP	= 0.40
Ratio directly connected to sewers, XIMP	= 0.01

(b) Urban Area 2

AREA	= 5 ha
TIMP	= 0.80
XIMP	= 0.80
SLOPE	= 0.0075

5. Obtain rainfall data

10-year storm: See Design Example 4.1 = 42.17 mm/h.
Hazel storm: use Design Chart 1.03

6. Prepare OTTHYMO input file. See Table (a) for the 10-yr storm.

7. Run OTTHYMO. The results for the 10-yr storm are summarized in Table (b).

10-yr storm, Q_{des10} = 4.52 m^3/s
Hazel storm, $Q_{deshazel}$ = 31.05 m^3/s

Task 2: Select a Suitable Ditch Cross Section

There may be several cross sections that can carry the design flows. The idea of Task 2 is to try various possible cross sections using a simple tool, the MTO Open Channel model, to examine the cross sections and select one that is the most preferred candidate for further work in Tasks 3 and 4. Use the following together with the calculated flow velocity and freeboard as a guide to select a cross section:

- The cross sectional area should be neither too large nor too small.
- The width:depth ratio of the ditch should be well proportioned, i.e. the ditch should be neither too deep nor too shallow and neither too wide nor narrow.
- The 10-yr velocity should not be too low to cause silting in the ditch, and the velocity under the Hazel storm condition should not be too high to require extraordinary erosion protection.

Table (a): OTTHYMO Input

```
2
* EXAMPLE 4.2 File Examp4_2.dat
*
START                    RAINFALL BEGINS AT 0 HRS
CHICAGO STORM            MET= 2 DUR= 4hr R= 0.38 DT= 10min
ICASE=2 (TIME,INTENSITY) =5,107.4 10,79.0
15,65.3 30,43 60,24.3 120,14.2 360,6.2
720,3.5 1440,2.0  -1
*
******************
* URBAN AREA # 1
******************
DESIGN STANDHYD ID=1  NHYD=101 DT=5 MINS AREA= 75HA
XIMP=0.25 TIMP= 0.40 DWF=0 LOSS= 1
SLOPE= 1% END= -1
*
ROUTE CHANNEL ID=2  NHYD=102  IDIN= 1 DT=5MIN
CHLGTH=1500m CHSLOPE= 0.75% FPSLOPE= 0.75%
VSN=1 NSEG= 3
(N,DIST)= 0.06,22  -0.035,35  0.06,57
(DIST,ELEV)= 0,100.0 12,97.0 22,97.0 26.5,95.5
30.5,95.5 35,97.0 45,97.0 57,100.0
****************
* RURAL AREA #2
****************
DESIGN NASHYD ID=3 NHYD=103 DT= 5MIN AREA= 220 ha DWF= 0
CN= 65  TP= 0.67hr -1
*
ADD HYD ID=4  NHYD= 104 ID= 2 + ID= 3
*
****************
*URBAN AREA #3
****************
*
DESIGN STANDHYD ID=5 NHYD=105 DT= 5MIN AREA = 5 HA
XIMP=0.80  TIMP=0.80 DWF=0 LOSS=1
SLOPE=0.75% END= -1
*
ADD HYD ID=6 NHYD=106 ID=4 + ID=5
*

FINISH
```

Table (b): OTTHYMO Output

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***** SUMMARY OUTPUT *****

Input filename: EXAMP4_2.DAT
Output filename: EXAMP4_2.OUT
Summary filename: EXAMP4_2.SUM
DATE: TIME:
USER:--
COMMENTS:---

****
** SIMULATION NUMBER 1**
****

W/E COMMAND	HYD	ID	DT min	AREA ha	Qpeak cms	Tpeak hrs	R.V. mm	R.C.	Qbase cms
START @ .00 hrs									
CHIC STORM [Ptot=33.93mm]			10.0						
DESIGN STANDHYD [I%=25.0:S%=1.00]	0101	1	5.0	75.00	3.81	1.58	14.88	0.44	.000
CHANNEL[1:0101]	0102	2	5.0	75.00	2.78	1.92	14.88	n/a	.000
DESIGN NASHYD [CN=65.0] [[N=3.0:Tp=0.67]	0103	3	5.0	220.00	2.03	2.25	6.21	0.18	.000
ADD [0102+0103]	0104	4	5.0	295.00	4.39	1.92	8.42	n/a	.000
*DESIGN STANDHYD [I%=80.0S%=0.75]	0105	5	5.0	5.00	0.82	1.50	27.76	0.82	.000
ADD[0104+0105]	0106	6	5.0	300.00	4.52	1.92	8.74	n/a	.000
FINISH									

=======================

For conciseness, this design example presents only the preferred cross section to show how to use MTO Open Channel model. For working details of the model, see the model's user manual.

1. Try a double trapezoidal cross section. The lower trapezoid is lined with riprap (D_{50} = 150 mm), and the upper trapezoid with grass.

   (a) Lower channel input parameters:
   Q_{des10} = 4.52 m^3/s
   Manning's n = 0.04 (for riprap) (Design Chart 2.01)

Bed slope, S_o = 0.0050 m/m
Side slope, Z = 4.0

Try the following channel width and depth.
b_w = 3.0 m
d_l = 1.0 m

Model output:
y = 0.67 m
ϑ_{10} = 2.24 kg/m^2
Q_{10} = 4.53 m^3/s

(b) Upper channel input parameters:
$Q_{deshazel}$ = 31.05 m^3/s
S_o = 0.0050 m/m
Manning's n = 0.05 (for grassed channel) (Design Chart 2.01)
Try b_{wul} = b_{wur} = 5 m (1/2 bottom width low flow channel)
Z_{ul} = Z_{ur} = 4 (side slope of upper channel)

Model output:
y = 1.50 m
ϑ_u = 2.12 kg/m^2 (upper trapezoid)
ϑ_l = 5.56 kg/m^2 (lower trapezoid)
Q = 31.1 m^3/s

Task 3: Detailed Hydraulic Analysis of the Preferred Cross Section

To calculate the water surface elevation corresponding to the preferred cross section and the design flow rate, use the U.S. Army Corps of Engineers HEC-2 computer model. For working details of the model, see the model's user manual. (The Windows version of HEC-2, called HEC-RAS, may also be used.)

Start the calculations at the downstream end of the ditch and with a known water surface elevation at this section. This water surface elevation, often called tailwater elevation, is normally that of the receiving watercourse and is determined through a hydraulic analysis of the receiving watercourse. Use of an arbitrarily assigned tailwater elevation may be erroneous. In this design example, it is assumed that the given tailwater elevations under the 10-yr and Hazel storm conditions have been established by a proper analysis of the receiving watercourse.

The water surface profile will be calculated for a number of cross sections over a length of approximately 220 m of downstream ditch area. Flow rates for both the 10-yr storm and Hazel storm will be modelled.

1. Prepare the HEC-2 input file, as shown in Table (c). Follow the guidance of HEC-2 user manual to select input values. Note the chosen values for the following data in the input file.

   - X Manning's n 0.04 (channel) 0.06 (flood plain)
   - X Expansion coef. 0.1
   - X Contraction coef. 0.3

2. Run HEC-2 and evaluate the output. Table (d) shows a summary of the output.

   (a) Submerged outlet.

   - X The depth of water in the ditch at the outlet is greater when the outlet is submerged than when the outlet is free flowing (not submerged). See Figure (b) for a comparison of the water surface profiles in the two cases. Use the water surface profile for the Hazel storm condition to determine the minimum amount of freeboard available.
   - X The rate of increase in water depth decreases in the upstream direction. The tailwater effect ceases at about 180 m upstream from the outlet for the 10-yr storm, and about 220 m for the Hazel storm.
   - X The flow velocity increases steadily in the upstream direction. The minimum velocity for the 10-yr storm is 0.36 m/s. Serious silting is not expected.

   (b) Free outlet.

   - X At the upstream sections, No. 1.00 to 2.20, the flow depth and velocity remain constant, as the flow becomes uniform.
   - X The 10-yr and Hazel storm flow velocities under the uniform flow condition will govern the design for channel lining in Task 4.
   - X At Sections No. 0.80 to 0.10 which are close to the outlet, the flow depth decreases and flow velocity increases marginally.
   - X At the ditch outlet, the flow becomes critical because of the free outlet condition. The maximum velocity of 2.38 m/s will be the governing condition for designing the erosion protection measures at the outlet as described in Task 4.

Table (c): HEC-2 Input

T1	STANDARD STEP CALCULATION								
T2	NON-STANDARD MTO DITCH								
T3	DRAINAGE MANAGEMENT MANUAL DESIGN EXAMPLE								
J1	0	2	0	0	0	1	0	0	101.25	0
J3	38	43	1	2	8	42	26	25		
NC	.06	.06	.040	.1	.3					
QT	4	4.52	31.05	4.52	31.05					
X1	0.00	10	5	50	0	0	0			
GR	104.0	0	104.0	5	101.0	17	101.0	22	100.0	26
GR	100.0	29	101.0	33	101.0	38	104.0	50	104.0	55
X1	0.01	10	5	50	10	10	10			
GR	104.05	0	104.05	5	101.05	17	101.05	22	100.05	26
GR	100.05	29	101.05	33	101.05	38	104.05	50	104.05	55
X1	0.20	10	5	50	10	10	10			
GR	104.10	0	104.10	5	104.10	17	101.10	22	100.10	26
GR	100.10	29	101.10	33	101.10	38	104.10	50	104.10	55
X1	0.30	10	5	50	10	10	10			
GR	104.15	0	104.15	5	101.15	17	101.15	22	100.15	26
GR	100.15	29	101.15	33	101.15	38	104.15	50	104.15	55
X1	0.40	10	5	50	10	10	10			
GR	104.20	0	104.20	5	101.20	17	101.20	22	100.20	22
GR	100.20	29	101.20	33	101.20	38	104.20	50	104.20	55
X1	0.50	10	5	50	10	10	10			
GR	104.25	0	104.25	5	101.25	17	101.25	22	100.25	26
GR	100.25	29	101.25	33	101.25	38	104.25	50	104.25	55
X1	0.60	10	5	50	10	10	10			
GR	104.30	0	104.30	5	101.30	17	101.30	22	100.30	26
GR	100.30	29	101.30	33	101.30	38	104.30	50	104.30	55
X1	0.80	10	5	50	20	20	20			
GR	104.40	0	104.40	5	101.40	17	101.40	22	100.40	26
GR	100.40	29	101.40	33	101.40	38	104.40	50	104.40	55
X1	1.00	10	5	50	20	20	20			
GR	104.50	0	104.50	5	101.50	17	101.50	22	100.50	26
GR	100.50	29	101.50	33	101.50	38	104.50	50	104.50	55
X1	1.20	10	5	50	20	20	20			
GR	104.60	0	104.60	5	101.60	17	101.60	22	100.60	26
GR	100.60	29	101.60	33	101.60	38	104.60	50	104.60	55
X1	1.40	10	5	50	20	20	20			
GR	104.70	0	104.70	5	101.70	17	101.70	22	100.70	26
GR	100.70	29	101.70	33	101.70	38	104.70	50	104.70	55
X1	1.60	10	5	5	0	20	20	20		
GR	104.80	0	104.80	5	101.80	17	101.80	22	100.80	26
GR	100.80	29	101.80	33	101.80	38	104.80	50	104.80	55
X1	1.80	10	5	50	20	20	20			
GR	104.90	0	104.90	5	101.90	17	101.90	22	100.90	26
GR	100.90	29	101.90	33	101.90	38	104.90	50	104.90	55
X1	2.00	10	5	50	20	20	20			

Table (c): HEC-2 Input (Contd.)

GR	105.0	0	105.0	5	5	17	102.0	22	101.0	26
GR	101.0	29	102.0	33	33	38	105.0	50	105.0	55
X1	2.20	10	5	50	50	20	20			
GR	105.10	0	105.10	5	5	17	102.10	22	101.10	26
GR	101.10	29	102.10	33	33	38	105.10	50	105.10	55
EJ										
T1	LOW FLOW CALCULATION									
T2										
T3	0	3	0	0	0	1	0	0	102.25	0
J1	2									
J2										
T1	CRITICAL FLOW FOR 10-YR									
T2										
T3										
J1	0	4		0	-1	1			101.00	
J2	3									
T1	CRITICAL FLOW HAZEL									
T2										
T3										
J1	15	5		0	-1	1			102.00	
J2										
ER										

Task 4: Erosion Protection Design

1. For the normal section of the channel, try using riprap with D_{50} = 50 mm.
   Critical shear stress, $\vartheta_c = 0.0642 * D_{50} = 9.6$ kg/m^2 > 5.56 kg/m^2 O.K. (5.31)

   Note: D_{50} = 150 mm is the smallest practical size of riprap. Therefore, it is not necessary to consider a smaller riprap size even though the shear force (ϑ_l) is almost half ϑ_c.

2. For the ditch outlet, also try riprap with D_{50} = 150 mm.
   At max. velocity, flow depth = 1.27 m (Table (d))
   Hydraulic radius, R, of channel at max. vel = Area/wetted perim. = 0.76 m
   For ϑ_c = 9.6 kg/m^2 and 1/R = 1.32,
   Permissible velocity = 3.4 m/s > max. vel. 2.38 m/s. O.K. (Design Chart 2.24)

Table (d): HEC-2 Output Summary

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Interactive Summary Printout
for MS/PC-DOS Micro Computers
September 1990

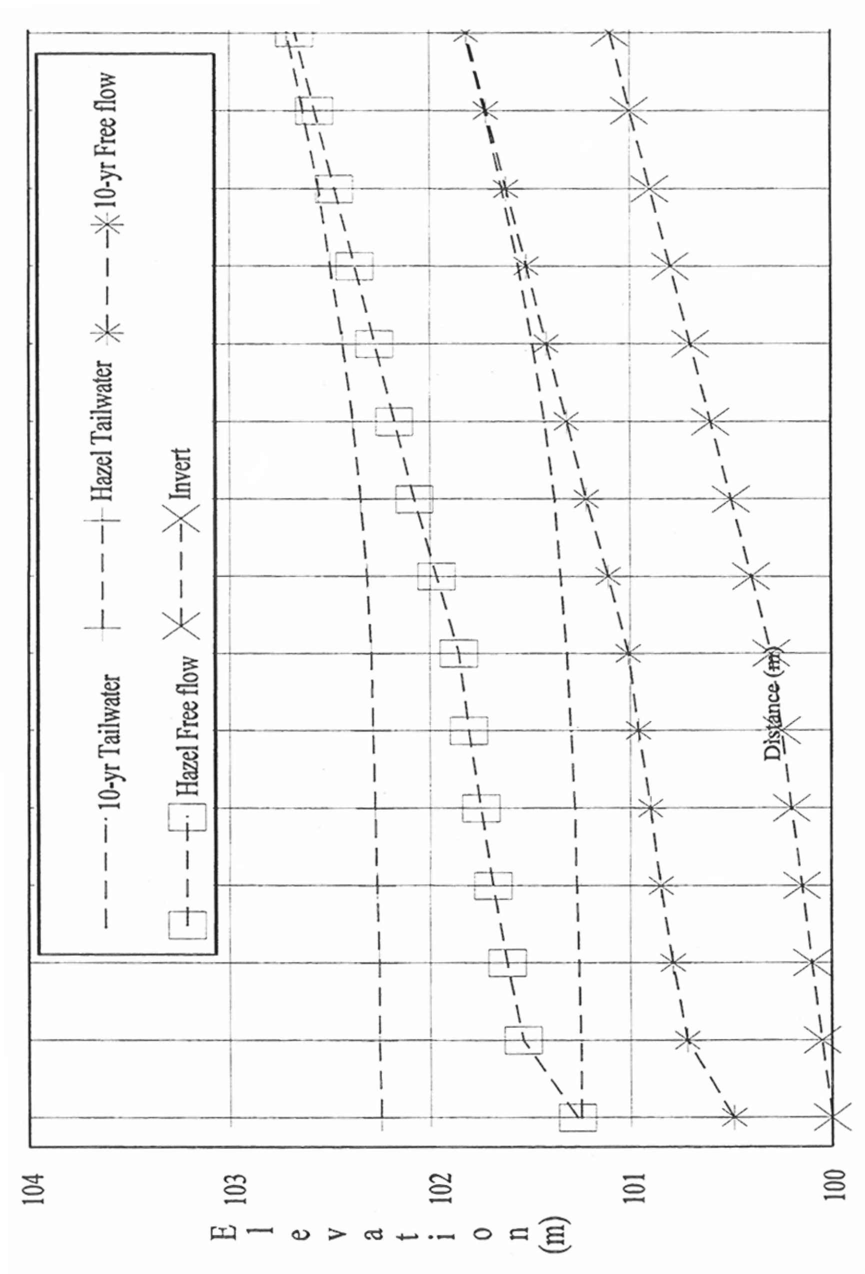
Note - Asterisk (*) at left of profile number
indicates message in summary of error list

SECNO	Q	CWSEL	CRIWS	DEPTH	ELMIN	VCH
.00	4.52	101.25	.00	1.25	100.00	.36
.00	31.05	102.25	.00	2.25	100.00	.79
*.00	4.52	100.49	.00	.49	100.00	1.86
*.00	31.05	101.27	.00	1.27	100.00	2.38
.10	4.52	101.25	.00	1.20	100.05	.39
.10	31.05	102.26	.00	2.21	100.05	.81
*.10	4.52	100.72	.00	.67	100.05	1.19
*.10	31.05	101.54	.00	1.49	100.05	1.70
.20	4.52	101.26	.00	1.16	100.10	.43
.20	31.05	102.26	.00	2.16	100.10	.84
.20	4.52	100.79	.00	.69	100.10	1.14
.20	31.05	101.62	.00	1.52	100.10	1.63
.30	4.52	101.27	.00	1.12	100.15	.47
.30	31.05	102.27	.00	2.12	100.15	.87
.30	4.52	100.85	.00	.70	100.15	1.12
.30	31.05	101.69	.00	1.54	100.15	1.60
.40	4.52	101.28	.00	1.08	100.20	.52
.40	31.05	102.28	.00	2.08	100.20	.90
.40	4.52	100.90	.00	.70	100.20	1.10
.40	31.05	101.75	.00	1.55	100.20	1.58
.50	4.52	101.30	.00	1.05	100.25	.57
.50	31.05	102.29	.00	2.04	100.25	.94
.50	4.52	100.96	.00	.71	100.25	1.09
.50	31.05	101.81	.00	1.56	100.25	1.56
.60	4.52	101.32	.00	1.02	100.30	.62
.60	31.05	102.30	.00	2.00	100.30	.97
.60	4.52	100.01	.00	.71	100.30	1.09
.60	31.05	101.86	.00	1.56	100.30	1.56
.80	4.52	101.35	.00	.95	100.40	.69
.80	31.05	102.32	.00	1.92	100.40	1.04
.80	4.52	100.11	.00	.71	100.40	1.08
.80	31.05	101.97	.00	1.57	100.40	1.53

Table (d): HEC-2 Output Summary (Contd.)

SECNO	Q	CWSEL	CRIWS	DEPTH	ELMIN	VCH
1.00	4.52	101.38	.00	.88	100.50	.78
1.00	31.05	102.35	.00	1.85	100.50	1.12
1.00	4.52	100.22	.00	.72	100.50	1.07
1.00	31.05	102.08	.00	1.58	100.50	1.52
1.20	4.52	101.43	.00	.83	100.60	.88
1.20	31.05	102.39	.00	1.79	100.60	1.19
1.20	4.52	101.32	.00	.72	100.60	1.07
1.20	31.05	102.18	.00	1.58	100.60	1.52
1.40	4.52	101.49	.00	.79	100.70	.95
1.40	31.05	102.44	.00	1.74	100.70	1.26
1.40	4.52	101.42	.00	.72	100.70	1.07
1.40	31.05	102.28	.00	1.58	100.70	1.51
1.60	4.52	101.56	.00	.76	100.80	1.00
1.60	31.05	102.50	.00	1.70	100.80	1.32
1.60	4.52	101.52	.00	.72	100.80	1.07
1.60	31.05	102.38	.00	1.58	100.80	1.51
1.80	4.52	101.64	.00	.74	100.90	1.03
1.80	31.05	102.56	.00	1.66	100.90	1.37
1.80	4.52	101.62	.00	.72	100.90	1.07
1.80	31.05	102.48	.00	1.58	100.90	1.51
2.00	4.52	101.73	.00	.73	101.00	1.05
2.00	31.05	102.64	.00	1.64	101.00	1.42
2.00	4.52	101.72	.00	.72	101.00	1.07
2.00	31.05	102.58	.00	1.58	101.00	1.51
2.20	4.52	101.83	.00	.73	101.10	1.05
2.20	31.05	102.72	.00	1.62	101.10	1.45
2.20	4.52	101.82	.00	.72	101.10	1.07
2.20	31.05	102.68	.00	1.58	101.10	1.51

Figure (b): Comparison of Water Surface Profiles



Design Example 4.3
Non-standard Ditch for Conveyance and

Required

Design a non-standard roadside ditch with a rock check dam across the ditch to convey the minor flow and to settle suspended sediment by retaining some of the runoff .

Given

X The drainage area and highway, as shown in Figure (a).
X Design for conveyance of the flow from the 10-yr storm. Use the AES precipitation data for the Pearson International Airport.
X Design the storage capacity based on the runoff volume from the 2-yr storm and for a duration equal to the time of concentration.
X The ditch invert slope is 0.25% along the downstream length of 0.5 km.

Preliminary Discussion

This design example shows the method for the design of a check dam if one is considered necessary, but the inclusion of this design example is not an endorsement of using check dams in roadside ditches for sedimentation. There is no known literature which reports statistical findings of long-term effectiveness of a check dam used as a sedimentation device.

In the highway environment, use of check dams in roadside ditches should be carefully balanced with considerations for safety of errant vehicles and long-term maintenance requirements of the check dams and associated ditches. For roadside safety considerations, consult highway engineers.

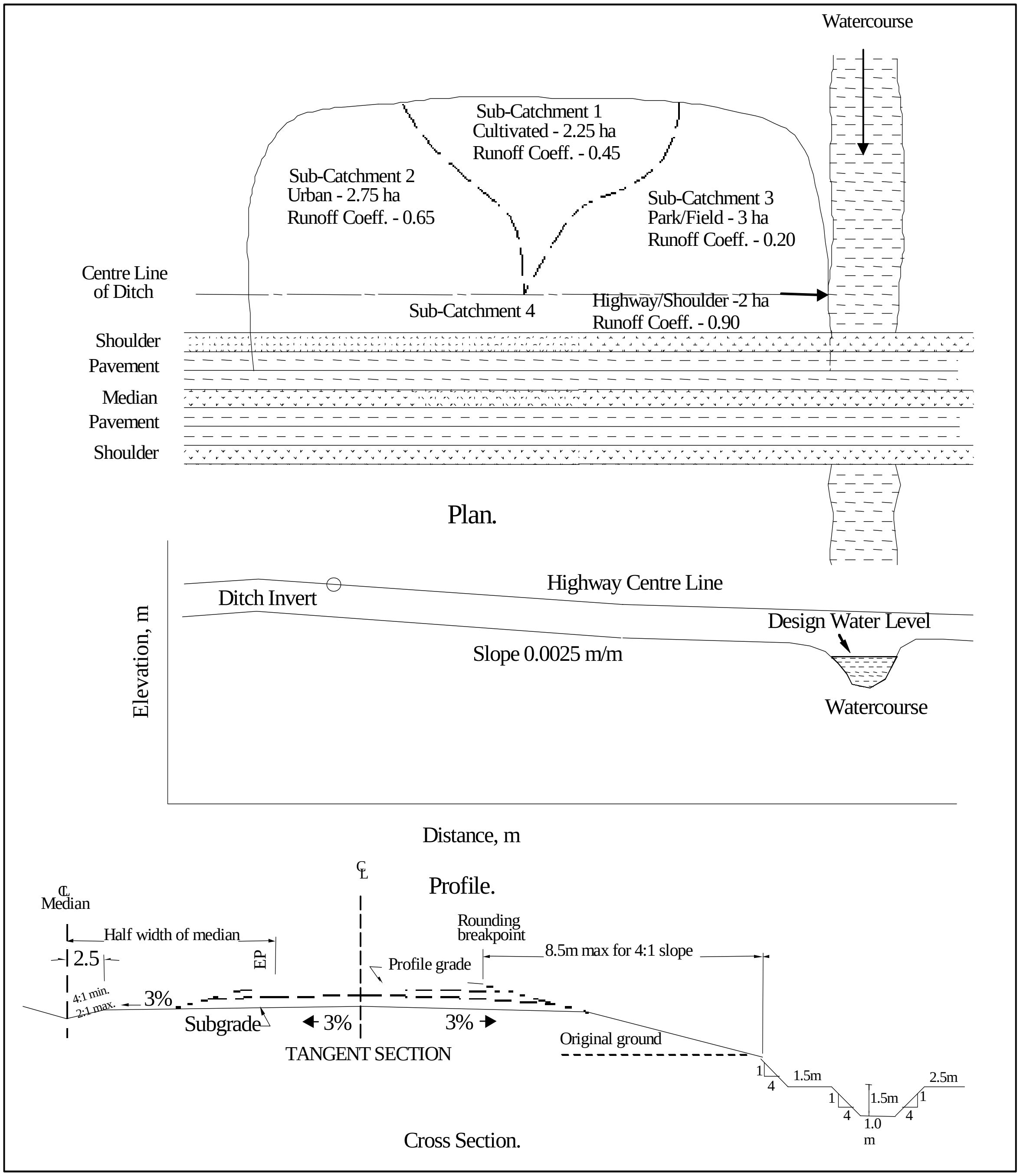
A decision to install a check dam in a roadside ditch is normally made at the preliminary design stage by the project manager or engineer.

Method

1. Determine the design flow, using Method 2 of Design Example 4.1. The results are:

2-yr intensity, I_2	= 34.74 mm/h
10-yr intensity, I_{10}	= 54.19 mm/h
Q_{des2}	= 0.50 m^3/s
Q_{des10}	= 0.79 m^3/s

Figure (a): Site of Design Example 4.3



2. Design the ditch size, using MTO Open Channel model. For working details of the model, see the model's user manual.

Assume a trapezoidal ditch with a bottom width of 1.0 m, depth 1.0 m, and side slope 4:1. Assume the ditch to be lined with riprap for erosion protection for the 10-yr flow.

2-yr Storm:

Input to model:

b_w = 1.0 m
d_l = 1.0 m
Q_{des2} = 0.5 m^3/s
n = 0.035
S_o = 0.0025 m/m
Z_{ll} = Z_{lr} = 4.0

Model results:

Q_2 = 0.5 m^3/s
y = 0.375 m (Note: This is the required depth)
A = 0.94 m^2
W_p = 4.09 m
R = 0.23 m
V_{max} = 0.53 m/s
F_r = 0.4 (the flow is subcritical).

10-yr Storm:

Input to model:

Q_{des10} = 0.79 m^3/s. The other input values are the same as for 2-yr storm.

Model results:

Q = 0.79 m^3/s
y = 0.462 m
A = 1.32 m^2
W_p = 4.81 m
R = 0.27 m
V_{max} = 0.60 m/s
F_r = 0.4 < 1.0

3. Design the check dam height for 2-yr storm.

Assume the duration of the storm = time of concentration, T_c (a basic principle of the Rational method).

T_c = 0.535 h (For calculation method, see Design Example 4.1) (8.15)

Total runoff volume = $Q_{des2} * T_c$

= 0.5 * 0.535 * 60 * 60 = 963 m^3

4. Design the storage volume behind the rock check dam by trial and error.

- X Try a height of rock check dam = 1.0 m
- X Determine the horizontal length of the triangle formed by the dam, the detained water surface, and the ditch longitudinal invert slope. This length is the shorter of:
  - (a) the total length of the ditch, 0.5 km, and
  - (b) the length of the detained water surface

    = dam height / invert slope

    = 1.0/0.0025 = 400 m. (Use this length)
- X Depth of detained water at centroid of triangle

  = 2/3 * dam height = 2/3 * 1.0 = 0.67 m
- X Width of detained water surface at centroid of triangle

  = Bottom width + 2 * water depth * side slope H:V ratio

  = 1.0 + 2 * 0.67 * 4 = 6.36 m
- X Cross sectional area of detained water at centroid

  = (1.0 + 6.36) * 0.67/2 = 2.44 m^2
- X Storage volume created by dam

  = 2.44 * 400 = 977 m^3 > 963 m^3 (runoff volume). O.K.

5. Calculate the flow depth over the check dam for the 10-yr storm.

Flow overtopping check dam

= Q_{des10} - Q_{des2} = 0.79 - 0.50 = 0.29 m^3

Assume the top of the dam acts as a broad crested weir with the maximum water depth over the check dam = H_{max}

Try a dam height of 1.0 m, with a crest length, L = 9.0 m

H_{max} = $(0.29 / (1.856 * 9.0))^{2/3}$ (8.74)

= 0.04 m

Required ditch height at the dam

= Dam height + water depth over dam + freeboard

= 1.0 + 0.04 + 0.3 = 1.34 m

Storm Sewers*

Detail Design Considerations

Give similar consideration to those listed in the previous section on Roadside Ditches, where applicable. Consider also the following additional points.

Sewer Materials

If there is no need for a particular type of sewer pipe material, specify in the design that the contractor may choose an alternative material. Specify the conditions for the choice. Essential conditions normally require the alternative pipe material not to be inferior to that called for in the original design. These conditions may include the following:

- No change to the original design hydraulic capacities and flow velocities under various design conditions.
- No shortening of the expected life cycle of the sewer. The sewer life expectancy is generally taken to be 50 years. There is no standard value for it.
- No change in the resistance to abrasion, corrosion and other forms of deteriorations assumed in the original design.
- Pipe joints will satisfy the requirements with respect to leakage proofing, durability and performance throughout the expected life cycle of the sewer.
- No added difficulty in making connections of catchbasins and other sewers and ditches with the sewer.
- No change in the structural strength requirements of the sewer.
- No increase in construction and maintenance costs to the sewer owner.

The proponent for the alternative pipe material should be required to provide evidence that the specified requirements are met.

If there is a need for a particular pipe material, for environmental, hydraulic or abrasion control reasons for example, clearly indicate this restriction the contractor must comply with on the contract drawing.

* This section is a continuation of Step 5 of the Detail Design Process.

Sewer Elevations

The following points should be accounted for in the detail design.

- Provide adequate ground cover to a sewer to protect it from frost damage. See Design Chart 4.03.
- Sewer inverts must be low enough to allow all connections to the sewer to be made with positive gradients. If tile drains in farming areas or basement drains in urban areas are connected to the storm sewer, these connections may be very deep.
- The sewer must have an adequate gradient to maintain a velocity to minimize sedimentation. Under the design flow condition, this velocity is about 0.75 m/s for smooth pipes and 0.9 m/s for corrugated pipes.

Hydraulic Conditions in the Sewer

Normally, for the design flow a sewer should be designed to:

- flow nearly full (90% full gives the best hydraulic efficiency);
- be subcritical gravity flow (not flowing under a hydrostatic head); and
- have a free outfall (no tailwater at the outfall).

However, other hydraulic conditions may occur and should be checked for. For example:

- the flow may be supercritical or critical on steep gradients; and
- the flow may occur under hydrostatic pressure due to an undersized pipe downstream or the outlet may be submerged by tailwater.

Sewer Outlet and Outlet Conditions

The following points should be checked:

- Whether the sewer has a proper outlet and whether the outlet has the required capacity under all design conditions.
- Whether the discharge of the sewer under certain conditions may greatly increase the flow or raise the water surface profile of the receiving drain or stream. (If this happens, the flows of other sewers and ditches that discharge to the same receiver maybe upset, and drainage problems may occur in the areas served by those other sewers and ditches).
- Whether erosion protection and energy dissipation is required.

Exfiltration to Road Subgrade

If a sewer alignment is underneath or close to a roadway, consider specifying the type of pipe joints trenching, pipe-laying, bedding and backfilling methods to ensure that exfiltration from the sewer will not occur. Consult geotechnical, pavement, and construction professionals as necessary. Exfiltration may induce movement of fine soil of the subgrade to cause partial differential settlement of the subgrade. In the worst situation, cavities may be formed in the subgrade and may cause sudden subsidence of the roadway. The result can be disruptive to traffic and injuries or fatalities. Similar precaution is needed if a sewer is on or near an embankment.

Safety to People and Vehicles

Inlets and outlets of storm sewers, especially large sewers, should be located outside the safe recovery area of the right-of-way, where possible. If this is not possible, provide safety guiderails or traversable grates in consultation with road designers. Traversable grates can be used only if they do not reduce the hydraulic capacity of the sewer and do not cause clogging (AASHTO, 1992).

Inlets and outlets of large storm sewers can be an attractive nuisance and a safety hazard to children. Fencing or grates should be provided to prevent entry.

Sewer Accessories

- Use standard details whenever possible.
- Provide end treatment (e.g. wing walls) to large inlets and outlets to smooth flow transition.
- A sewer connection to a maintenance hole (M.H.) should flow in the same general direction as the main sewer and should have a positive gradient, generally not flatter than 0.015 m/m.
- A sewer that discharges over a slope should provide erosion protection for the slope.
- Provide a maintenance hole in the following situations:

    - where two or more sewers meet;
    - where pipe size changes;
    - where an abrupt change in alignment occurs;
    - where an abrupt change in invert grade occurs; and
    - at intermediate points along tangent sections at 100 - 150 m for sewers less than 1200 mm dia. and 200 - 300 m for larger sewers.

Alternative Design of Details

Possible alternatives are generally limited to choice between pipe materials, and trade-offs between pipe gradients and sizes.

Design Example 4.4
Storm Sewer Network

Required

Design a storm sewer network for highway pavement drainage to convey the flows from the minor storms.

Given

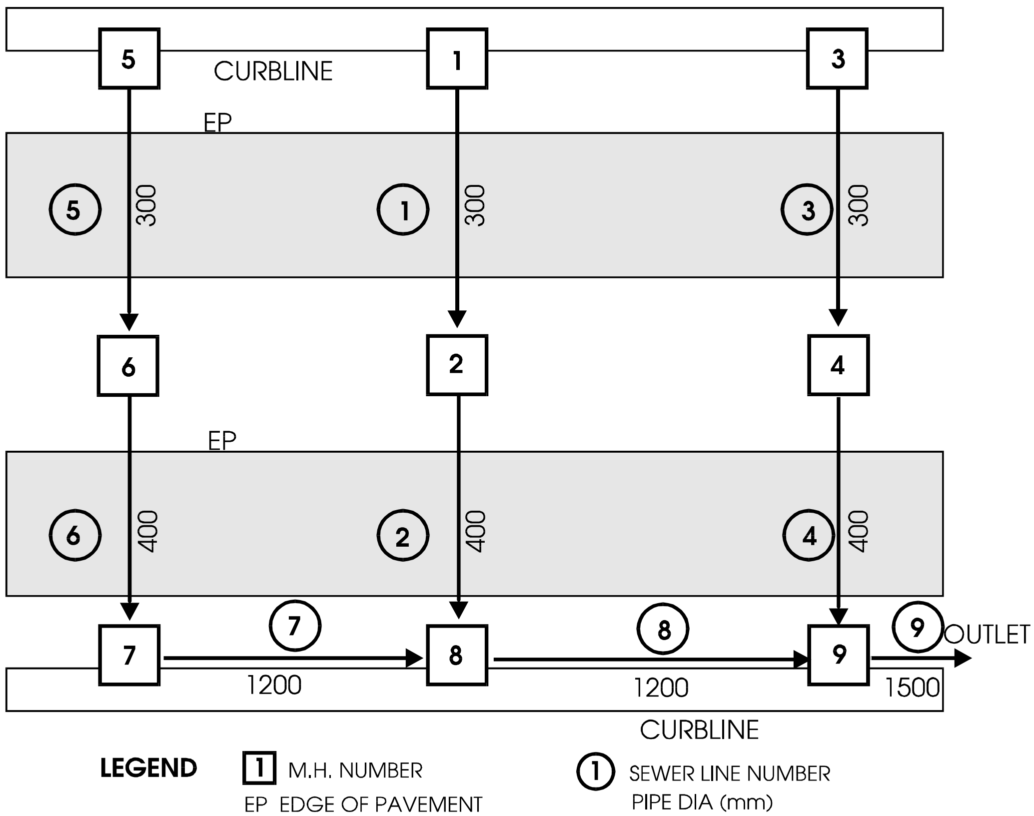
- The storm sewer network plan, as shown in Figure (a). The numeric data are summarized below.
- Design for the 10-yr storm. Use the AES precipitation data for Pearson International Airport.

Pipe ID	Length (m)	Roughness	Area (ha)	Runoff Coef.	Inlet Time (min)
1	20	0.012	0.75	0.70	15
2	20	0.012	0.75	0.70	15
3	20	0.012	0.75	0.70	15
4	20	0.012	0.75	0.70	15
5	20	0.012	0.75	0.70	15
6	20	0.012	0.75	0.70	15
7	125	0.012	0.75	0.70	15
8	125	0.012	0.75	0.70	15
9	200	0.012	0.00		15

Preliminary Discussion

- The calculation starts at the uppermost point of the sewer network and proceeds downstream. It is convenient to have the calculated results tabulated. A sample blank form is provided in Table 4B.1 in Appendix 4B.
- A trial and error procedure may be needed to determine the pipe sizes and elevations.
- The inlet time for a M.H. is the time of concentration of the flow from the furthest point in the direct catchment of the M.H. to the M.H. It is usually assumed to be 15 min.
- Two methods are provided: (1) design charts; (2) MTO Storm Sewer model.

Figure (a): Sewer Network Diagram



Method 1 (Using Design Charts)

Method 1 is a manual method aided by design charts. It is provided as a review of the first principle of design. The calculated results are shown in Table (a).

1. Use the Rational method, Equation 8.19, to estimate the runoff from the direct catchment. The time of concentration for a M.H. is the greater of two values: its inlet time or the cumulative time of concentration from the uppermost M.H. to the current M.H. If two sewers converge at a M.H., the time of concentration is the greater for the values of the two sewers.

2. The total design flow to be conveyed by a pipe is equal to the runoff from the M.H. direct catchment plus the flow converging to this M.H. from other upstream pipes, if any.

3. A pipe is sized to convey the design flow, using Manning equation (8.66). Normally, a pipe is designed for free flow (that is, no tailwater) condition at the downstream end of the pipe.

M.H. 1 to M.H. 2 (Pipe # 1)

Col. 1 to 4. Enter the data for the first sewer pipe on line 1

Col. 5 Catchment area, A = 0.75 ha

Col. 6 Runoff coef., C = 0.70

Col. 7 A * C = 0.525

Col. 8 Cum (A * C) = 0.525
Col. 7 of this line + Col. 8 of preceding line (= 0 for first line).

Col. 9 Cum T_c at M.H. 1 = 15 min (= Inlet time at M.H. 1)

Col. 10 Rain intensity, I = 92.7 mm/h
(See Design Example 4.1 for method for determining I.)

Col. 11 Design flow, Q_{des} = 0.0028 * Col. 8 * Col. 10 (8.19)
= 0.0028 * 0.525 * 92.7 = 0.136 m^3/s

Col. 12 Pipe length, L = 20 m

Col. 13 Try a pipe invert slope, S_o= 0.025 m/m

Note: S_o is usually selected nearly equal to the road grade to minimize trench excavation, but it may be varied for pipe hydraulic reasons.

Col. 14 Determine D by trial and error, using Design Chart 2.29.

Q_{des} = 0.136 m^3/s
S_o = 0.025
Manning's n = 0.012
Try D = 0.28 m
Pipe area, A_p= 0.0616 m^2
Hydr. radius = 0.07 m
Q = $0.0616 * 0.07^{0.667} * 0.025^{0.5} / 0.012 = 0.137\ m^3/s$

Note: Since the trial D = 0.28 is smaller than the minimum pipe size, use minimum D = 300 mm.

Table (a): Sewer Network Design Calculation

HWY. NO. 101

STORM SEWER DESIGN CALCULATIONS

DESIGN FREQUENCY 10 yr

W.P. NO. Ex6.5

RAINFALL STATION(S) AES PEARSON INT'L AIRPORT

DESIGNED BY: SSA DATE:

DESIGNED 'n' 0.012 (CONC)

CHECKED BY: DATE:

MINIMUM COVER 1.5 M

LOCATION				DRAINAGE AREA				RUNOFF			PIPE SELECTION							PROFILE				REMARKS
FROM MH or CBMH No.	TO Sta.	FROM MH or CBMH No.	TO Sta.	A ha	C	A*C	Cumul. A*C	Cumul. Tc (Ti = 15) min	i mm/h	Q m3/s	Pipe L m	Pipe So m/m	Pipe D m	Actual Capacity (full) m3/s	V (full) m/s	Time of Flow min	Minor Losses m	Fall in Sewer m	Upstream Surface Elev. m	Upstream InvEL m	D/S InvEL m	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
1	1+000	2	1+020	0.75	0.70	0.53	0.53	15	92.7	0.136	20	0.025	0.30	0.17	2.34	0.14	0.0	0.5	98.40	96.60	96.10	
2	1+020	8	1+040	0.75	0.70	0.53	1.05	15.14	92.1	0.27	20	0.025	0.40	0.36	0.84	0.12	0.0	0.5	100.80	95.97	95.47	
3	1+000	4	1+020	0.75	0.70	0.53	0.53	15	92.7	0.136	20	0.02	0.30	0.15	0.09	0.16	0.0	0.4	100.80	95.10	94.70	
4	1+020	5	1+040	0.75	0.70	0.53	1.05	15.16	92.0	0.27	20	0.02	0.40	0.32	2.53	0.13	0.0	0.4	100.80	94.57	94.17	
5	1+000	6	1+020	0.75	0.70	0.53	0.53	15	92.7	0.136	20	0.015	0.30	0.13	1.18	0.18	0.0	0.3	100.80	98.30	98.00	
6	1+020	7	1+040	0.75	0.70	0.53	1.05	15.18	92.0	0.27	20	0.015	0.40	0.28	2.2	0.15	0.0	0.3	100.80	97.87	97.57	
7	2+000	8	2+125	0.75	0.70	0.53	1.58	15.33	91.3	0.04	125	0.011	1.20	4.43	3.92	0.53	0.4	1.4	100.80	96.16	94.76	
8	2+125	9	2+250	0.75	0.70	0.53	3.15	15.86	89.2	0.79	125	0.009	1.20	4.00	3.54	0.59	0.3	1.1	99.20	94.38	93.28	
9	2+250	OUTLET		0.75	0.70	0.53	4.73	16.45	86.9	1.15	200	0.005	1.50	5.10	3.06	1.09	0.1	1.0	98.10	91.91	91.91	

Col. 15 Q_{full} (D = 0.3 m) = 0.17 m^3/s

Col. 16 V_{full} (D = 0.3 m) = Q_{full} /area = 2.34 m/s

Col. 17 Time of flow from M.H. 1 to 2 = pipe length / velocity of flow
= 20 / (2.34 * 60) = 0.14 min

Note: In theory, the velocity used in this calculation should be the actual velocity for D = 0.3 m and Q = 0.136 m^3/s. However, the approximate velocity of 2.34 m/s is acceptable.

Col. 18 For M.H. 1, enter zero.

Col. 19 Fall in sewer= Col. 12 * Col. 13 = 0.025 * 20 = 0.50 m

Col. 20 Enter surface elev. = 98.40 m (Data from survey or highway design.)

Col. 21 Upstream inv elev. = Crown elev. of incoming sewer
- head loss - diameter of outgoing pipe
= Col. 22 previous line + Col. 14 previous line
- Col. 18 this line - Col. 14 this line

Note: For the first M.H., allow a sufficient depth of cover (say 1.50 m).

Upstream invert elev. = 98.40 - 1.5 - 0.3 = 96.60 m

Col. 22 Downstream invert elev. = 96.60 - 0.5= 96.1 m

M.H. 2 to M.H. 8 (Pipe # 2)

Col. 1 to 7 Follow the same procedure as for M.H. 1 to M.H. 2.

Col. 8 Cum A * C = Cum A * C at M.H. 1 + A * C for M.H. 2
= Col. 8 (previous line) + Col. 7
= 0.525 + 0.525 = 1.05

Col. 9 T_C = Col. 9 (previous line) + Col. 17 (previous line)
= 15.00+ 0.14 = 15.14 min

Col. 10 to 17 Follow the same procedure as for M.H.1 to M.H.2.

Col. 18 Calculate the transition loss at M.H. 2 due to the increase in the pipe size from 0.3 m to 0.4 m. Use a transition loss coefficient of 0.2 a for gradual expansion.

At M.H. 2 : (For full flow)

Velocity in pipe #1 = 2.34 m/s
Velocity in pipe #2 = 2.84 m/s

$$h = \frac{0.2\ (2.842 - 2.342\ )}{2g} = 0.027\ \text{m}$$

Col. 19 Follow the same procedure as for M.H. 1 to M.H. 2.

Col. 20 Enter the surface elevation at M.H. 2.

Col. 21 Enter the upstream invert elevation for pipe #2
= 96.10 + 0.3 - 0.03 - 0.4 = 95.97 m

Col. 22 Enter the downstream invert elevation for pipe #2
= 95.97 - 0.50 = 95.47 m

M.H. 5-6-7-8 and M.H. 3-4-9

Follow the same procedure as for M.H. 1-2-8. The results are shown in Table (a).

M.H. 8 to M.H. 9 (Pipe # 8)

Col. 1 to 7, 10 to 17 and 19 are completed the same way as for Pipe # 1.

Col. 8 Cum. A * C = Col. 8 (line 2) + Col. 8 (line 7) + Col. 7
= 1.05 + 1.575 + 0.525 = 3.15

Col. 9 To calculate T_c, use the greater of T_c for the main pipe and T_c for the lateral pipe.

For the main pipe : T_c = Col. 9 (line 2) + Col. 17 (line 7)
= 15.33 + 0.53 = 15.86 min

For the lateral pipe : T_c = Col. 9 (line 2) + Col. 17 (line 2)
= 15.14 + 0.12 = 15.26 min

Use T_c = 15.86 min.

Col. 18 Calculate junction loss, h_8 at M.H. 8.

From Table (a), obtain the flow rate and velocities through Pipes # 2, 7 and 8:

Q_2 = 0.27 m^3/s

Q_7 = 0.40 m^3/s
Q_8 = 0.79 m^3/s
V_2 = 3.12 m/s
V_7 = 2.43 m/s
V_8 = 2.75 m/s

Note: V_2, V_7 and V_8 shown above are velocities calculated for the design flows, using Manning equation. Approximate velocities as shown in Table (a) for full flow of pipes may be used instead. The errors should be small in most cases.

From Design Chart 4.02:
K_7 = 0.02
K_2 = 1.32
h_8 = $[0.768 * 2.72^2 - 0.392 * 2.42^2 (1 - 0.02) - 0.271 * 3.12^2 (1 - 1.32)]$
$/ (2 * 0.768 * 9.81) = 0.283$ m

Col. 19 to 22 Follow the same steps presented for the previous pipe.

Note: Junction losses for M.H. 9 can be similarly calculated.

Method 2 (Using MTO Storm Sewer Model)

Most of the input data required for the model are the same as for Method 1. Additional requirements are pipe diameters and up/downstream pipe elevations of each pipe. If the hydraulic grade line calculation is required, the ground elevations at all M.H. and the outlet water surface elevation are required. For guidance on the working details of the model, see the model's user manual.

Table (b) shows the input file for the model. The results are shown in Table (c). The model actually creates four output files. Table (c) has been compiled from these four files.

Table (b): MTO Storm Sewer Model Input

```
          MTO Storm Sewer  Mode 1                         Version 2.0

PIPE DATA
No. OF PIPES =     9
PIPE                                INVERT
 ID  LENGTH SIZE   ROUGNESS   AREA  RUNOFF COEF    UP     DOWN   TC
 No.   (m)  (mm)                   (ha)                    (m)     (m)    (min)
   1   20   300        .012       .75          .70        96.60   96.10 15.0
   2   20   400        .012       .75          .70        95.97   95.47 15.0
   3   20   300        .012       .75          .70        95.10   94.70 15.0
   4   20   400        .012       .75          .70        94.57   94.17 15.0
   5   20   300        .012       .75          .70        98.30   98.00 15.0
   6   20   400        .012       .75          .70        97.87   97.57 15.0
   7  125  1200        .012       .75          .70        96.16   94.76 15.0
   8  125  1200        .012       .75          .70        94.38   93.28 15.0
   9  200  1500        .012       .00          .70        92.91   91.91 15.0
MANHOLE/CATCHBASIN DATA
No. OF MH/CB  =    9
  MHCB   D/STREAM    OUTFLOW  INFLOW PIPES    UPSTREAM
  ID       MHCB         PIPE       ID                   MHCB ID
  No.      ID              ID  No.    No.1 No.2 No.3    No.1 No.2 No.3
   1        2               1
   2        8               2           1          1
   3        4               3
   4        9               4           3          3
   5        6               5
   6        7               6           5          5
   7        8               7           6          6
   8        9               8           7     2    7          2
   9       10               9           8     4    8          4
EXTERNAL CONTRIBUTING AREA
No. of external areas =      0
CATCH    EXTERNAL  DRAINAGE
BASIN    AREA      COEF    TC
 ID      (ha)            (min)
Rainfall Source: 4
 AES STATION TORONTO PEARSON APT
Number of Return Periods: 1
              10
HYDRAULIC GRADELINE DATA
No. OF MH/CB ALONG MAIN SEWER LINE          5
ELEV HGL AT OUTLET (m)                 94.00
MH/          TYPE   GROUND  INFLOW   PIPE      LATERAL PIPES
JUNCT                  ELEV      ID         DEFL     ID  DEFL   ID  DEFL
No.                     (m)      No.        (deg)    No.  (deg)   No. (deg)
  9          MHCB      98.10   8            0       4    90     0    0
  8          MHCB      99.20   7            0       2    90     0    0
  7          MHCB     100.80   6           90       0     0     0    0
  6          MHCB     100.80   5            0       0     0     0    0
  5          MHCB     100.80   0            0       0     0     0    0
END
```

Table (c): Compiled MTO Storm Sewer Model Output

Return Period = 10 Years

HYDRAULIC ANALYSIS FOR PROPOSED SYSTEM

MH/CB No.	Outflow Pipe (mm)	In-Flow (m^3/s)	Flow Capacity (m^3/s)	Actual Flow (m^3/s)	Flow capacity (%)	Flow Depth (m)	Actual Velocity (m/s)	Remarks
1	300	.13	.17	.13	80	.20	2.61	
2	400	.27	.36	.27	75	.26	3.12	
3	300	.13	.15	.13	89	.22	2.37	
4	400	.27	.32	.27	83	.28	2.84	
5	300	.13	.13	.13	104	.30	1.81	Surcharged
6	400	.27	.28	.27	96	.32	2.50	
7	1200	.39	4.47	.39	8	.24	2.42	
8	1200	.77	3.96	.77	19	.36	2.72	
9	1500	.98	5.41	.98	18	.43	2.33	

==

Return Period = 10 Years

HYDRAULIC ANALYSIS FOR OPTIMIZED SYSTEM

MH/CB No.	Outflow Pipe (mm)	In-Flow (m^3/s)	Flow Capacity (m^3/s)	Actual Flow (m^3/s)	Flow capacity (%)	Flow Depth (m)	Actual Velocity (m/s)	Remarks
1	300	.13	.17	.13	80	.20	2.61	
2	400	.27	.36	.27	75	.26	3.12	
3	300	.13	.15	.13	90	.22	2.37	
4	400	.27	.32	.27	84	.28	2.84	
5	400	.13	.28	.13	48	.20	2.18	
6	400	.27	.28	.27	97	.32	2.50	
7	500	.39	.43	.39	90	.37	2.49	
8	700	.77	.94	.77	81	.48	2.72	
9	800	.98	1.01	.98	96	.63	2.29	

HYDRAULIC GRADE LINE ANALYSIS

Return Period = 10 Years

MHCB/ JUNCT No.	OUTFLOW PIPE No.	GROUND ELEV (m)	DESIGN FLOW (m^3/s)	MINOR LOSSES Hf (m)	Hend (m)	Hm (m)	TOTAL LOSSES (m)	HGL ELEVATION DOWN (m)	UP (m)
9	9	98.10	.978	1.003	.276	.021	1.300	95.28	95.30
8	8	99.20	.768	1.096		.283	1.379	96.40	96.68
7	7	100.80	.391	1.401		.494	1.896	98.08	98.58
6	6	100.80	.269	.299		.243	.542	98.87	99.12
5	5	100.80	.128	.300		.251	.551	99.42	99.67

NOTES:

Minor losses from entrances (He), exits (Hexit) and bends (Hb) are included as manhole/junction (Hm) losses for surcharged conditions.

Return Period = 10 Years

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STORM SEWER DESIGN CALCULATIONS

LOCATION MH No.	DMH No.	DRAINAGE AREA(ha) A	C	AC	CUM AC	RUNOFF CUM TC (min)	I (mm/h)	Q (m^3/s)	D (m)	PIPE SELECTION So (m/m)	Qcap (m^3/s)	V (m/s)
1	2	.75	.70	.52	.52	15.1	92.3	.134	300	.025	.17	2.34
2	8	.75	.70	.52	1.05	15.3	91.5	.269	400	.025	.36	2.84
3	4	.75	.70	.52	.52	15.2	91.9	.134	300	.020	.15	2.09
4	9	.75	.70	.52	1.05	15.3	91.5	.269	400	.020	.32	2.54
5	6	.75	.70	.52	.52	15.2	91.9	.134	300	.015	.13	1.81
6	7	.75	.70	.52	1.05	15.3	91.5	.269	400	.015	.28	2.20
7	8	.75	.70	.52	1.57	15.9	89.0	.391	1200	.011	4.47	3.95
8	9	.75	.70	.52	3.15	16.4	87.1	.768	1200	.009	3.96	3.50
9	10	.00	.70	.00	4.20	17.5	83.2	.978	1500	.005	5.41	3.06

Pavement Drainage*

The Function of Pavement Drainage

Pavement drainage is intended to convey the minor flow (usually 2 to 10-year design storm) from the road pavement and shoulders, sidewalks and adjacent parking lots to a drain effectively. Good pavement drainage is essential to maintaining traffic efficiency and safety in wet weather. Water on the pavement slows down traffic. The splash of water by a passing vehicle reduces the visibility of the drivers of this vehicle and nearby vehicles. A water film on the pavement may cause loss of contact between the pavement and vehicle tires, a phenomenon known as hydroplaning. Free-standing puddles can be dangerous to passing vehicles because of the resulting torque exerted on the vehicles. (U.S. Federal Highway Administration, 1984).

Elements of Pavement Drainage

The elements of a pavement drainage system include:

- gutters with and without curbs;
- gutter inlets and catchbasins; and
- receiving ditches or storm sewers.

Curbs and Gutters

A curb confines the surface water from the roadway and adjacent areas to the gutter. The gutter behaves as a flat channel which transports the collected surface water to an inlet. The gutter usually has a smoother surface texture than the pavement, so it is hydraulically more efficient. If the gutter is full, some of the water may spread to a narrow strip of the pavement along the gutter. The total width of water on this strip and the gutter is called the *spread* and its permissible width is specified as a design criterion. It is based on balancing the costs of pavement drainage with the risks and delay to vehicular traffic and inconvenience and possible hazard to pedestrian traffic.

Curbs and gutters are used mainly on highways in urban areas. Gutters are also used along with concrete median barriers. In rural areas, curbs and gutters are generally not used because of costs; the highway pavement drains in sheet flow toward the shoulder and into the roadside ditch.

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* This section is a continuation of Step 5 of the Detail Design Process.

Figure 4.3 shows the types of curbs and gutters most frequently used by MTO. Their hydraulic capacities are shown in Design Charts 4.04 to 4.13. For construction details of standard gutters, see Ontario Provincial Standard Drawings, OPSD series 600.

Gutter Inlets

Figure 4.4 shows the types of gutter inlets most frequently used by MTO. Their hydraulic capacities are shown in Design Charts 4.14 to 4.18. A combination of an overflow side-weir and a grate inlet may be used at road sags and at locations where litter-clogging is a persistent problem. For construction details of standard gutter inlets, see OPSD 400 series.

Roadway Characteristics Affecting Pavement Drainage

In addition to the elements of pavement drainage mentioned above, the following roadway characteristics affect pavement drainage.

Longitudinal Grades

Generally, the longitudinal grades of the pavement and its gutter are the same. A suitable longitudinal grade is essential for the water to flow down-slope along the gutter if the pavement is curbed. The minimum grade to make a gutter hydraulically functional is about 0.3% and should not be less than 0.2% in very flat terrain. Minimum grades in very flat terrain can be maintained by use of a rolling gutter profile or by warping the cross fall to create the necessary gutter rolling profile. In road sags, a minimum grade of 0.3% within 15 m on each side of the lowest point of the sag is recommended for hydraulic reasons (U.S. Federal Highway Administration, 1984). On uncurbed pavement, a minimum longitudinal grade is not essential if there is sufficient cross fall to enable the surface water to flow across the pavement toward the roadside ditch.

Cross Fall

Insufficient Cross Fall The design of pavement cross fall is often a compromise between the need for reasonably steep cross falls for drainage and relatively flat cross falls for driver comfort. Available literature (U.S. Federal Highway Administration, 1979; AASHTO, 1991 under revision) reports that cross falls of 2% have little effect on friction demand for vehicle stability, or driver effort in steering, especially with power steering. Cross falls for driving quality is specified in *Geometric Design Standards for Ontario Highways* (Ontario Ministry of Transportation, 1985) and for drainage performance in directives. If a conflict occurs between these two criteria, some remedial measure in the design that will mutually satisfy both criteria is necessary.

Figure 4.3: Curbs and Gutters

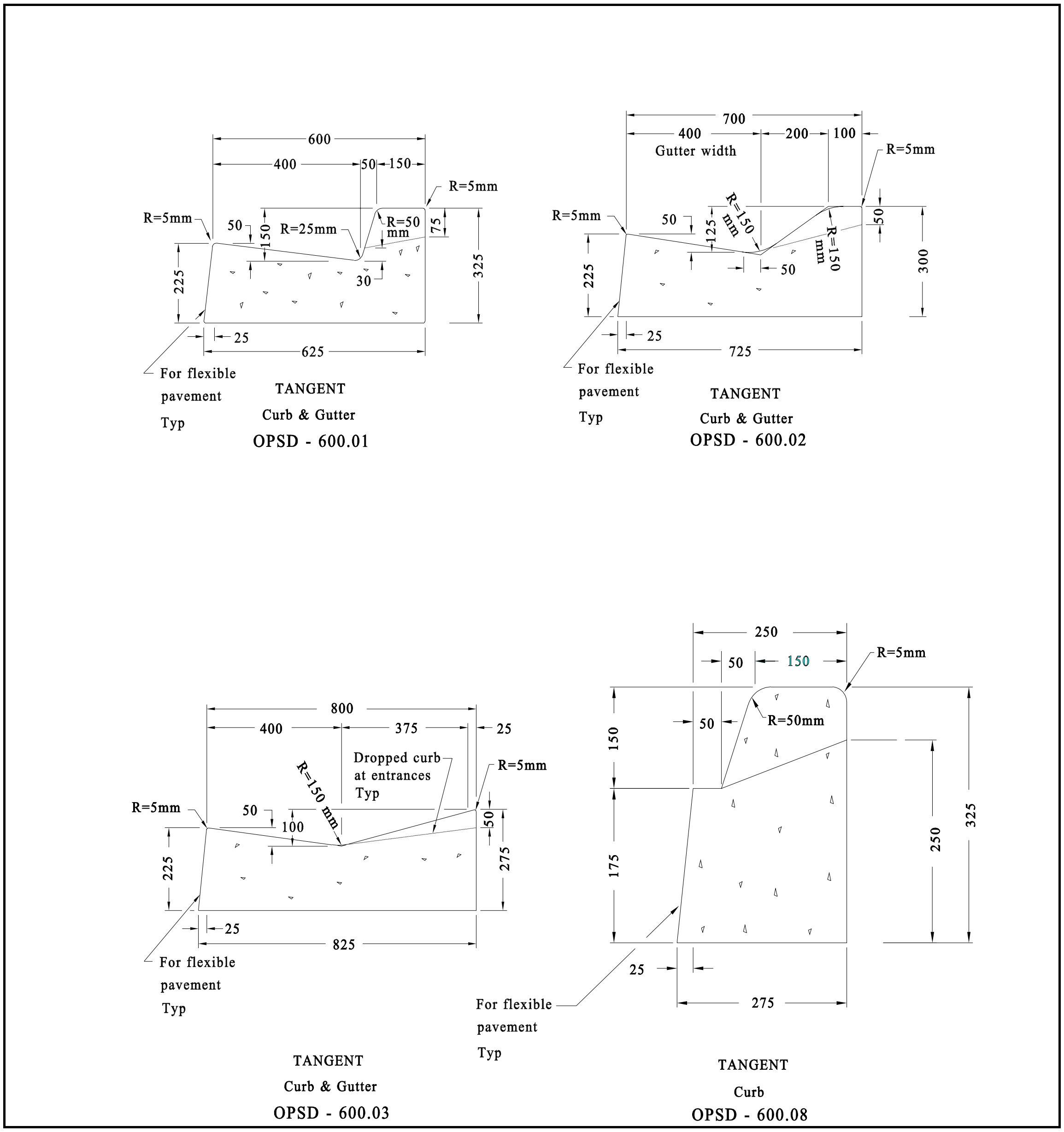
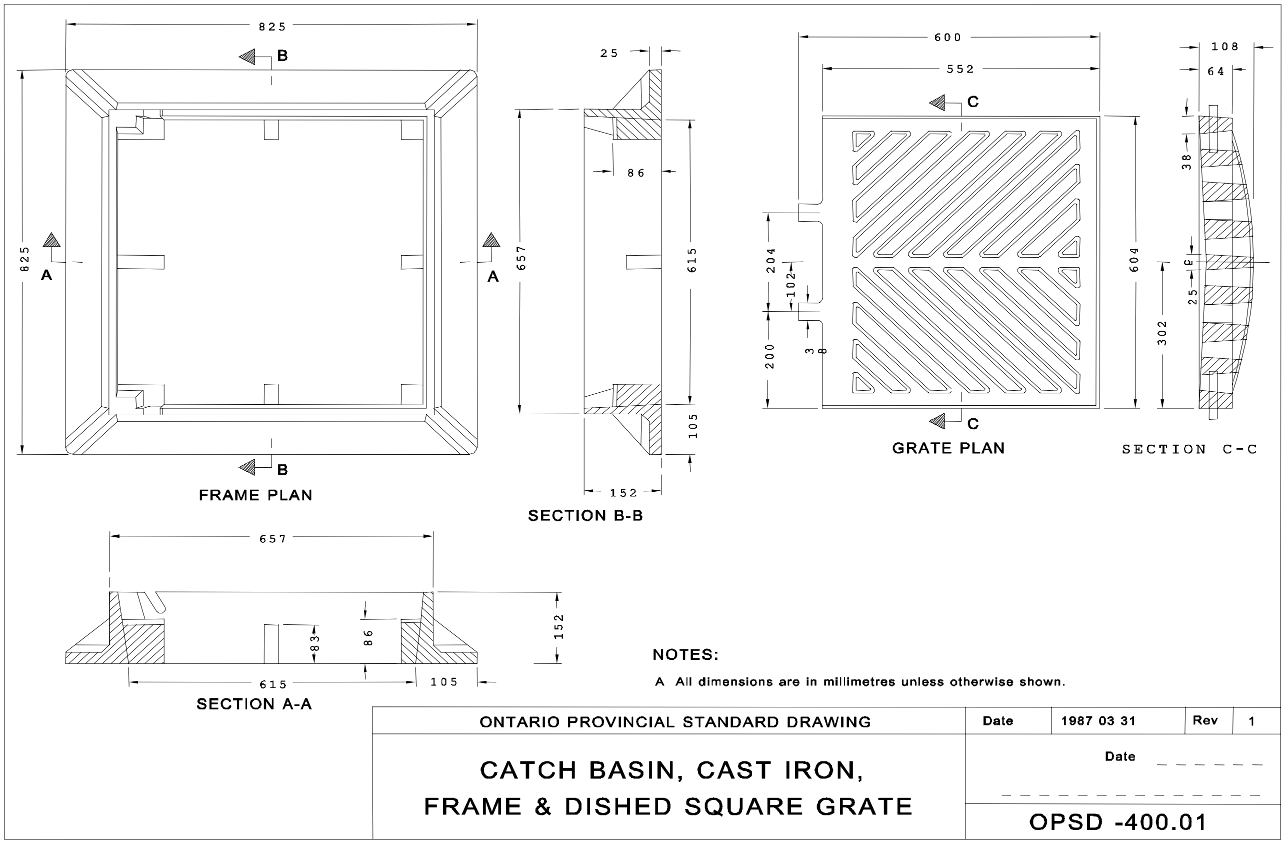


Figure 4.4: Gutter Inlets



Three or More Lanes Where three or more lanes are inclined in the same direction on multi-lane pavements, it is desirable that each successive pair of lanes outward from the first two lanes from the crown line, have an increased slope. This is an effective measure in reducing water depth on pavements. The two adjacent lanes adjacent to the crown line should be pitched at the normal slope, and successive lane pairs outward should be increased by about 0.5% to 1%. Where three or more lanes are provided in each direction, the maximum pavement cross slope should be limited to 4% (AASHTO, 1991 under revision), or the limit specified in *Geometric Design Standards for Ontario Highways*, whichever is the lesser.

It is desirable to provide a break in cross fall at two lanes, with three lanes the upper limit. Although not widely encouraged, inside lanes can be sloped toward the median. This should not be used unless four continuous lanes or some physical constraint on the roadway elevations occurs, since inside lanes are used for high speed traffic and the allowable water depth is lower. Where practical, median areas should drain to a median gutter or ditch which can be connected by a cross-road storm sewer to a main sewer or roadside ditch.

Road Shoulders Road shoulders should generally be sloped to drain away from the pavement, except with raised, narrowed medians.

Cross Fall Transitional Areas A careful check should be made of designs to minimize the number and length of flat pavement sections in cross fall transition areas, and consideration should be given to increasing cross falls in sag vertical curves, crest vertical curves, and in sections of flat longitudinal grades. Where gutters are located in shoulder or median areas, depressed gutters sections can be effective at increasing gutter capacity and reducing spread on the pavement (AASHTO, 1991 under revision).

Pavement Texture

Pavement texture is an important consideration for pavement drainage. Although the hydraulic designer will have little control over the selection of the pavement type or its texture, it is important to know that pavement texture does have an impact on the buildup of water depth on the pavement during rain storms. A good macro-texture provides a channel for water to escape from the tire-pavement interface and so reduces the potential for hydroplaning.

Hydroplaning

It is generally agreed that hydroplaning is not a factor affecting pavement drainage. Nor is it practical for a designer to produce a pavement drainage design that results in significant reduction in hydroplaning. The primary factors that affect the potential for hydroplaning conditions are:

- vehicle speed;
- tire conditions (pressure and tire tread);
- pavement texture;
- roadway geometrics;
- pavement conditions (rutting, depression, roughness); and
- the spread of water on the roadway.

In many respects, hydroplaning is analogous to iced or snow conditions. A Texas Transportation Institute study concludes that "it is the driver responsibility to exercise prudence and caution when driving during wet conditions." (AASHTO, 1996)

Median Barriers

Where continuous median barriers are installed, it is necessary to provide inlets (and gutters if preferred) to collect the water which drains toward the barrier and connect the inlets to a storm sewer. If a median barrier cuts off the major overland flow, however, the pavement drainage system will not be able to convey this flow. In this situation, a continuous median barrier should not be used and some other alternative type of barrier should be considered, such that the major flow may pass across the highway. Another possible alternative is to provide a major flow roadside ditch on the upstream side of the highway. This issue should be addressed in the preliminary design stage.

Shoulder Gutters

Shoulder gutters may be used to protect slopes, especially fill slopes, from erosion caused by water from the pavement. Normally, shoulder gutters are required on fill slopes (2:1) higher than 6 m and on fill slopes higher than 3 m if the highway grade is greater than 2%. Special design provision may be needed depending on the fill material and slope erosion protection design. Work with geotechnical designers of the slopes as necessary for a satisfactory solution.

External Tributary Drainage Areas

It is good practice to intercept runoff from external drainage areas by a separate gutter (or ditch) and inlets, if the flow will otherwise drain toward the pavement. Examples of these external areas are adjacent slopes, roadside rest areas and landscape plots.

Gutter Flow Calculation

Gutter flow is shallow flow in an "open channel". The Manning equation can be modified for use in calculating gutter flow. For an example of basic calculation of gutter flow, see Design Example 4.5.

Inlet Spacing

For normal pavement sections, the spacing between two consecutive inlets is determined by the amount of water that the second (downstream) inlet will receive. The spacing should be such that the maximum spread immediately upstream of the second inlet does not exceed the permissible spread. If the calculated spacing is greater than the desirable distance for maintenance purpose (usually 150 m), the design spacing will be set to the nominal spacing required for maintenance.

Additional inlets are normally provided at the following locations:

- sag points in the gutter grade;
- upstream of median breaks, entrance and exit ramp gores, cross walks, and street intersections;
- immediately upstream and downstream of bridges;
- immediately upstream of cross slope reversals (e.g. at superelevations);
- on side streets at intersections away from possible pedestrian paths;
- at the end of channels in cut sections;
- behind curbs, shoulders or sidewalks to drain low lying areas; and
- where necessary to collect snow melt.

Design Examples of Pavement Drainage

Five design examples of pavement drainage are provided:

Design Example 4.5: Calculation of Gutter Flow
Design Example 4.6: Inlet Spacing Design, Typical Highway Segment
Design Example 4.7: Inlet Spacing Design, Superelevation
Design Example 4.8: Inlet Spacing Design, Road Sag
Design Example 4.9: Roadside Ditch Inlet Spacing Design

Design Example 4.5
Calculation of Gutter Flow

Required

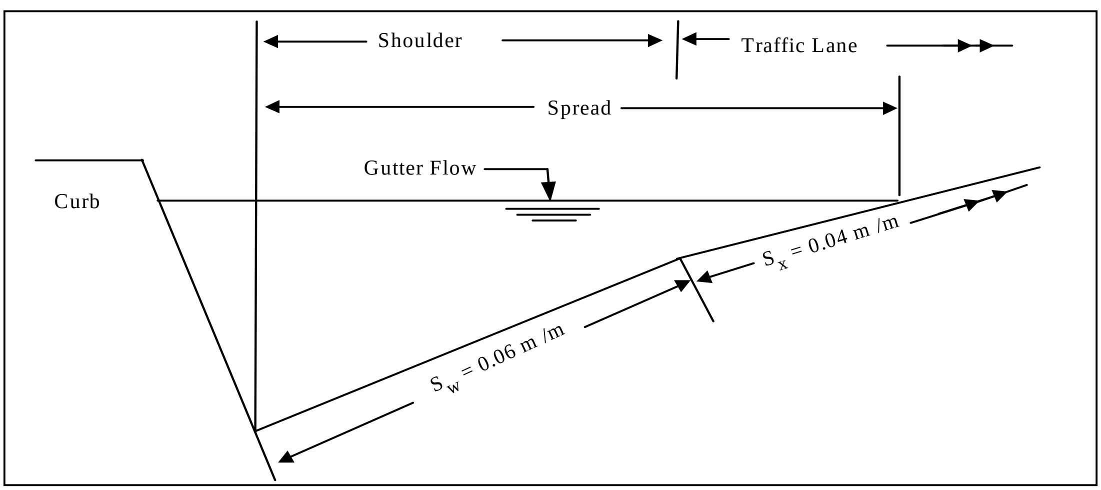
Calculate the capacity of a highway drainage gutter using the following two methods:

- X Method 1: from first principle.
- X Method 2: using design charts.

Given

- X A composite gutter, as shown in Figure (a), is used.
- X Longitudinal slope, S_o = 0.005 m/m
- X Manning's n = 0.015 (For conc. gutter/asphalt, rough finish) (Design Chart 2.01)
- X Cross slope, S_x = 0.04
- X Depressed slope, S_w = 0.06
- X Depressed width, W = 0.6 m
- X Spread, T = 3.0 m

Figure (a): Curb and Gutter Cross Section



Preliminary Discussion

This design example is intended to provide a review of the principle of gutter flow calculation for interested readers.

Manning equation is widely adopted for open channel flow and can only be applied to gutter flow with some modifications. This is because the hydraulic radius in Manning equation does not adequately describe shallow flow condition, which is the case with gutter sections that are characterised by very large spreads and small depths.

The following modified Manning equation is considered to be more applicable for gutter flow:

$$Q = 0.375 * S_o^{0.5} * d^{2.667} / (n * S_x) \qquad (4.1)$$

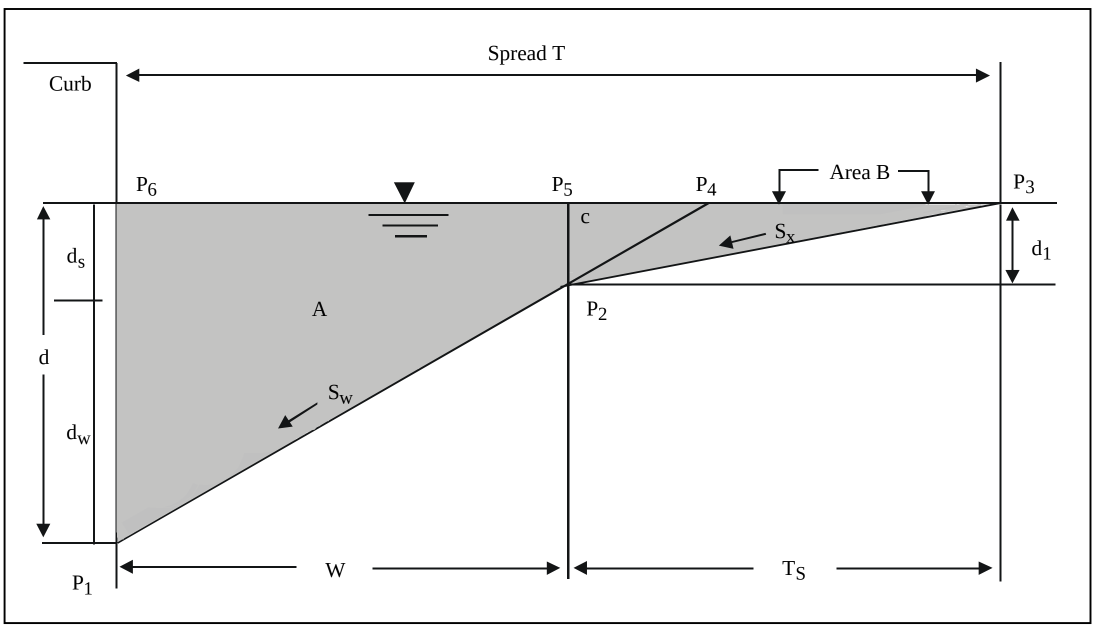
where:

Q = Gutter flow m^3/s
d = Depth of flow at curb, m

Method 1 (From First Principle)

For definition of symbols, see Figure (b).

Figure (b): Definition of Symbols



Calculate the depth of gutter flow, d.

$d_w = W * S_w$	$= 0.6 * 0.06$	$= 0.036$ m
$T_s = T - W$	$= 3.0 - 0.6$	$= 2.4$ m
$d_s = T_s * S_x$	$= 2.4 * 0.04$	$= 0.096$
$d = d_s + d_w$	$= 0.036 + 0.096$	$= 0.132$

2. Find the total gutter flow Q_{A+B}.

   (a) Divide the flow area into three parts: A ($P_1P_2P_5P_6$), B ($P_2P_3P_5$), and C ($P_2P_4P_5$).

   (b) Calculate the flow in triangular area (A + C).

   Q_{A+C} = $(0.375 * 0.005^{0.5} * 0.132^{2.667}) / (0.015 * 0.06) = 0.133\ m^3/s$ (4.1)

   (c) Calculate the flow in triangular area C.

   Q_C = $(0.375 * 0.005^{0.5} * 0.096^{2.667}) / (0.015 * 0.06) = 0.056\ m^3/s$ (4.1)

   (d) Q_A = $0.133 - 0.056 = 0.077\ m^3/s$

   (e) Calculate the flow in triangular area B.

   Q_B = $(0.375 * 0.005^{0.5} * 0.096^{2.667}) / (0.015 * 0.04) = 0.085\ m^3/s$ (4.1)

   (f) Determine the flow in total area A + B.

   Q_{A+B} = $0.077 + 0.085 = 0.162\ m^3/s$

   Note: If the design flow Q_{des} is given, the spread, T, can be calculated by the above method through a trial and error procedure: assume a value of T, calculate Q. Then compare Q with Q_{des}. If Q does not agree closely with Q_{des}, revise the assumed T and repeat the calculation.

Method 2 (Using Design Charts)

1. Calculate: W/T = 0.2 S_w/S_x = 1.5

2. Determine the flow in triangular area B.

   Q_B = $0.09\ m^3/s$ (Design Chart 2.28)

3. Find E_o (the ratio of Q_A/Q_{A+B}).

   E_o = 0.46 (Design Chart 4.22)

4. By definition of Design Chart 4.22,

   Q_{A+B} = $Q_B / (1 - E_o) = (0.09) / (1 - 0.46) = 0.167\ m^3/s$

Design Example 4.6
Inlet Spacing Design, Typical Highway Segment

Required

Design the inlet spacing for a highway segment with uniform cross fall and longitudinal grade on a straight horizontal alignment.

Given

- X The plan, profile, cross section and pavement types of a highway as shown in Figure (a).
- X There is no additional flow contribution from areas outside the roadway.
- X Design for the 10-yr storm. Rainfall intensity = 110 mm/h
- X Type of curb and gutter: OPSD 600.01. Grate inlet: OPSD 400.01.
- X Cross fall, S_x = 0.02 m/m, at Station 1000; and S_x = 0.04 m/m at Station 1800.
- X Longitudinal slope, S_o = 0.02 m/m, at Station 1000; S_o = 0.04 m/m at Station 1800.
- X Allowable spread, T = 1.9 m.
- X Allowable maximum inlet spacing = 150 m.

Preliminary Discussion

This design example uses the MTO CBSpace program to do the calculation. The design example will first show how to enter input data, then it will explain the underlying calculation method.

The MTO CBSpace program is developed in Lotus 1-2-3* macros (Release 5). Lotus 1-2-3 macros do not allow the use of the mouse. Therefore, use the arrow keys and HOME key to move the cursor. After typing a data item, **be sure to press ENTER**. For more information on MTO CBSpace, and user instructions, see the program's README.TXT.

If manual calculation is preferred, the design form in Table 4B.2 in Appendix 4B may be used.

Method

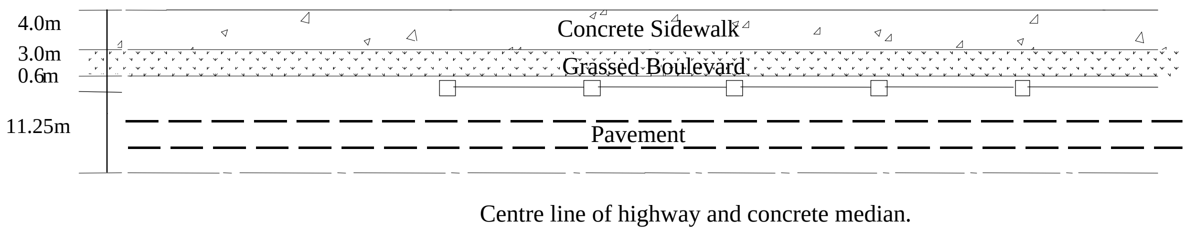
1. Start MTO CBSpace. The MenuPage appears as shown in Figure (b).

2. On the MenuPage, enter **1** for the curb and gutter type 600.01; **1** for the inlet type 400.01; and **2** for goto Project Panel. Press ENTER after each entry. The Project Panel appears as shown in Table (a).

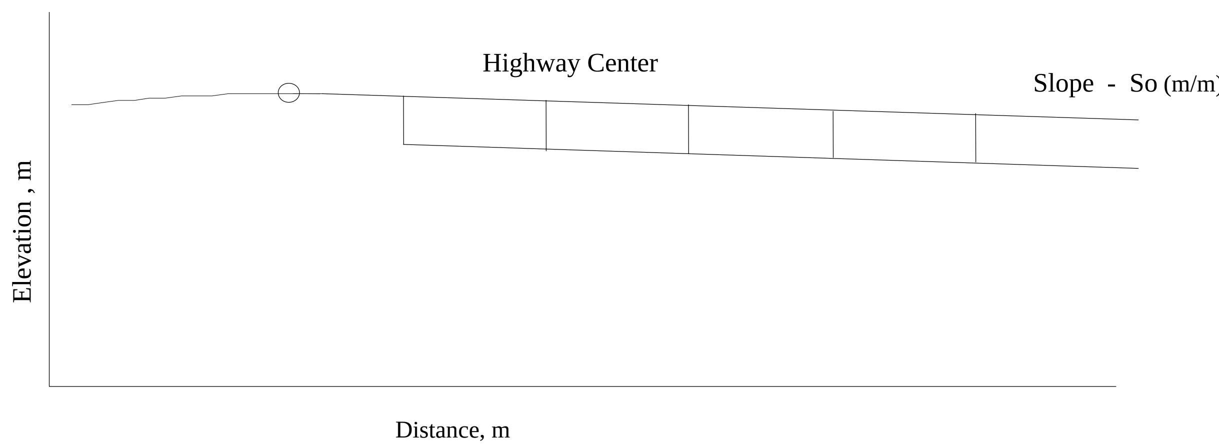
---

* Lotus 1-2-3 is a registered trademark of Lotus Development Corporation.

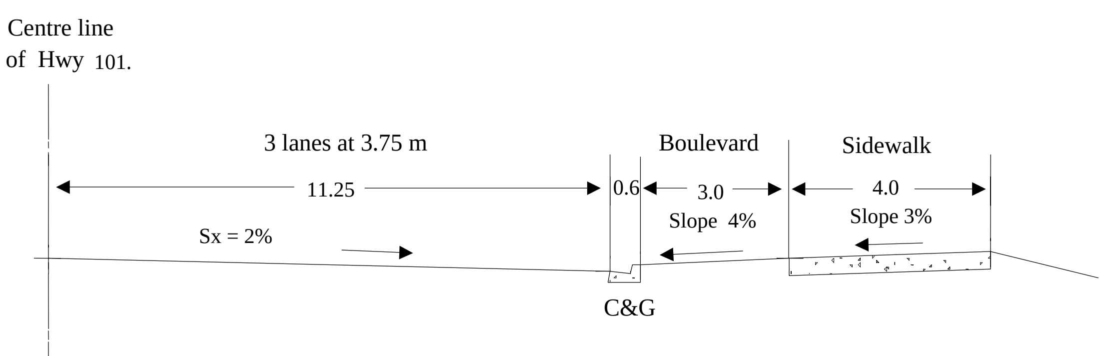
Figure (a): Site of Design Example 4.6



Plan



Profile.



Typical Half Cross-section.

Figure (b): MenuPage of MTO CBSpace

Menu Page	
Enter Curbs and Gutter Type Number: 1 = OPSD 600.01 2 = OPSD 600.02 3 = OPSD 600.03 [Use with OPSD 400.03]	>>
Enter Inlet Type Number: 1 = OPSD 400.01 2 = OPSD 400.03	>>
Goto: 1 = **New [Warning:** Will erase all data] 2 = Goto Project Panel 3 = Auto Spacing 4 = User - fixed Spacing 5 = Quit	>>
Instruction: Use arrow keys to move cursor to blue area. Type input. Press ENTER.	

3. On the Project Panel, enter values for Hwy No., ..., Rainfall intensity, ..., Inlet time.

4. Press the HOME key to move the cursor to the green cell at the top left corner of the Project Panel. Press ENTER. The MenuPage returns. Enter **3** in the bottom blue area to select Auto Spacing. The Project Panel re-appears. The cursor stops at Col. 2, Line 1 to wait for input.

   Note: Auto Spacing will determine the inlet spacing automatically, using the data to be entered in Steps 5 and 6 below. **Do not skip Step 4** to confuse the macros.

5. Enter the start station **1000.0** in Col. 2, Line 1. The cursor jumps to Col. 3, Line 2.

6. On Line 2, enter **11.25**, ..., **3.00** for the roadway widths W1, ..., W4 and the corresponding runoff coefficients, **0.95**, ..., **0.60** for C1, ..., C4. Enter gutter grade and cross fall. If a correction is necessary after an entry, move the cursor back to the cell to make the correction.

   Note: MTO CBSpace will calculate the values of Col. 13 to 19 once the cursor is at Col. 12 and the ENTER key is pressed. The cursor then stops at Col. 20 to wait for input.

7. Type comments in Col. 20 or simply press ENTER. MTO CBSpace automatically calculates the station value (1106.4) in Col. 2. Then it stops at Col. 3, Line 3 to wait for input.

8. Repeat Steps 6 and 7 for Lines 3 and 4.

   Note: By comparing the results of Lines 3 and 4, the designer will see that the spacing, L, in Line 4 has reached a constant and no further calculation is needed.

Table (a): Sewer Inlet Spacing - Design Example 4.6

(To goto MENUPAGE, move cursor to green cell, press ENTER)

Sewer Inlet Spacing Calculation

Storm Frequency (Yr) ____________
Rainfall Station ____________ -
Rainfall Intensity (mm/h) 110
Allowable Spread (m) 1.9
Allow. Max. Spacing (m) 150
Curb & Gutter Type OPSD 600.01
Inlet Type OPSD 400.01
Inlet Time (min) 15

Hwy No. ______________________________
W.P. No. ______________________________
Designed By __________ **Date** __________
Checked By __________ **Date** __________

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
		Inpyt Data										Calc.Data	Design Results						
		Area Widths, W (m)				Runoff Coef., C				Gutter Grade So (m/m)	Cross-fall Sx (m/m)	Com-posite C	Gutter Flow Qg (m3/s)	Local Runoff Qr (m3/s)	Inlet Spacing L (m)	Calc. Spread T (m)	Inlet Capacity Qi (m3/s)	Carry Over Qc (m3/s)	
		W1	W2	W3	W4	C1	C2	C3	C4										
1	1000.0																		
2	1106.4	11.25	0.60	4.00	3.00	0.95	0.95	0.95	0.60	0.02	0.02	0.89	0.0522	0.0522	106.4	1.9	0.0285	0.0267	
3	1161.3	11.25	0.60	4.00	3.00	0.95	0.95	0.95	0.60	0.02	0.02	0.89	0.0522	0.0285	54.9	1.9	0.0285	0.0267	
4	1216.2	11.25	0.60	4.00	3.00	0.95	0.95	0.95	0.60	0.02	0.02	0.89	0.0522	0.0285	54.9	1.9	0.0285	0.0267	
5																			
6																			
7																			
8	1800.0																		
9	1950.0	11.25	0.60	4.00	3.00	0.95	0.95	0.95	0.60	0.04	0.04	0.89	0.0779	0.0779	150.0	1.2	0.0315	0.0464	
10	2100.0	11.25	0.60	4.00	3.00	0.95	0.95	0.95	0.60	0.04	0.04	0.89	0.1243	0.0779	150.0	1.5	0.0443	0.0799	
11	2250.0	11.25	0.60	4.00	3.00	0.95	0.95	0.95	0.60	0.04	0.04	0.89	0.1578	0.0779	150.0	1.7	0.0488	0.1091	
12	2400.0	11.25	0.60	4.00	3.00	0.95	0.95	0.95	0.60	0.04	0.04	0.89	0.1870	0.0779	150.0	1.8	0.0532	0.1338	
13	2503.5	11.25	0.60	4.00	3.00	0.95	0.95	0.95	0.60	0.04	0.04	0.89	0.1875	0.0537	103.5	1.9	0.0576	0.1299	
14	2614.4	11.25	0.60	4.00	3.00	0.95	0.95	0.95	0.60	0.04	0.04	0.89	0.1875	0.0576	110.9	1.9	0.0576	0.1299	
15	2725.4	11.25	0.60	4.00	3.00	0.95	0.95	0.95	0.60	0.04	0.04	0.89	0.1875	0.0576	110.9	1.9	0.0576	0.1299	
16																			
17																			
18																			
19																			
20																			

9. Move the cursor to Line 8 to do the design for the highway segment starting with Station 1800.0.

   Note: Line 8 is chosen arbitrarily to provide some white space to separate the calculation of the previous highway segment. It will be just fine to choose Line 7 or 9 instead.

10. Repeat Steps 5 to 8 for Lines 8 to Line 15. At Line 15, the design spacing, L, reaches a constant. No further calculation is needed.

11. Press the HOME key and then press ENTER to bring back the MenuPage. Enter **5** in the bottom blue area to quit.

Explanation of the Calculation Method

Col 13:

Composite C = (W1 * C1 + W2 * C2 + W3 * C3 + W4 * C4) / (W1 + W2 + W3 +W4).

Col. 14:

MTO CBSpace uses the equations derived from hydraulic tests (Marsalek, 1982) to calculate gutter flow, Q_g.

Note: For manual calculations, Equation 4.1 may be used as explained in Design Example 4.5. An alternative manual method is to use Design Charts 4.04 to 4.13 which are based on the same equations derived from the hydraulic tests (Marsalek, 1982).

The following is an illustration for using the Design Charts:

For $S_o = 0.02$, $S_x = 0.02$, $T = 1.9$, and curb and gutter of OPSD 600.01,
Q_g = 0.055 m^3/s (Design Chart 4.04)

Col. 15:

Q_r is the runoff contributed by the roadway between the inlet under design and the previous (upstream) inlet.

$Q_r = Q_g - Q_c$ (of previous line of the spreadsheet).

Col. 16:

L is the distance of the roadway between the inlet under design and the previous (upstream) inlet.

$L = Q_r * 10000 / (0.0028 * (W1+W2+W3+W4) * \text{Rain intensity} * \text{Composite C})$ (8.19)

If L > Allowable maximum spacing (150 m in this design example), MTO CBSpace will set L to the allowable maximum spacing and recalculate Q_r and Q_g:

Q_r = 0.0028 * Composite C * Rain intensity * Allowable maximum spacing * (W1 + W2 + W3 + W4) / 10000 (8.19)

Q_g = Q_r (re-calculated) + Q_c (previous line)

Col. 17:

If L < Allowable maximum spacing, T = Allowable spread.
If L = Allowable maximum spacing, MTO CBSpace will re-calculate T by trial and error, using the equations mentioned in Col. 14.

Col. 18:

The inlet capacity, Q_i, is obtained from hydraulic laboratory test results for selected combinations of curb and gutter type, inlet type, S_x and S_o (Marsalek, 1982). MTO CBSpace uses lookup tables based on these test results.

Note: The same test results have been plotted as Design Charts 4.14 to 4.18.

Col. 19:

Q_c is the amount of runoff that the inlet cannot capture. This excess runoff is carried over to the next (downstream) inlet.
$Q_c = Q_g - Q_i$

Design Example 4.7
Inlet Spacing and Design, Superelevation

Required

Design the inlet spacing for a segment of a highway on a 45 degree horizontal curve.

Given

- The plan, profile and cross section of the highway as shown in Figure (a).
- There is no additional flow contribution from areas outside the roadway.
- Design for 10-yr storm. Rain intensity 110 mm/h.
- Type of curb and gutter: OPSD 600.01
- Type of grate inlet: OPSD 400.01
- Longitudinal slope, S_o = 0.05 m/m.
- Allowable spread, T = 1.9 m
- Allowable maximum inlet spacing = 150 m.
- Carry-over runoff, Q_c, at the entry point of curve from the upstream segment of the roadway = 0.01 m^3/s
- Highway geometric design data:

  - Starting station 3150.
  - Superelevation revolves about curb line.
  - Tangent runout section: 30 m; S_x of median lane changes from 0.02 m/m to 0.
  - Spiral transition: 85 m; S_x of median lane changes from 0 to 0.06 m/m; S_x of the other two lanes changes from 0.02 to 0.06 m/m.
  - Circular curve: 236 m; S_x remains constant at 0.06 m/m.

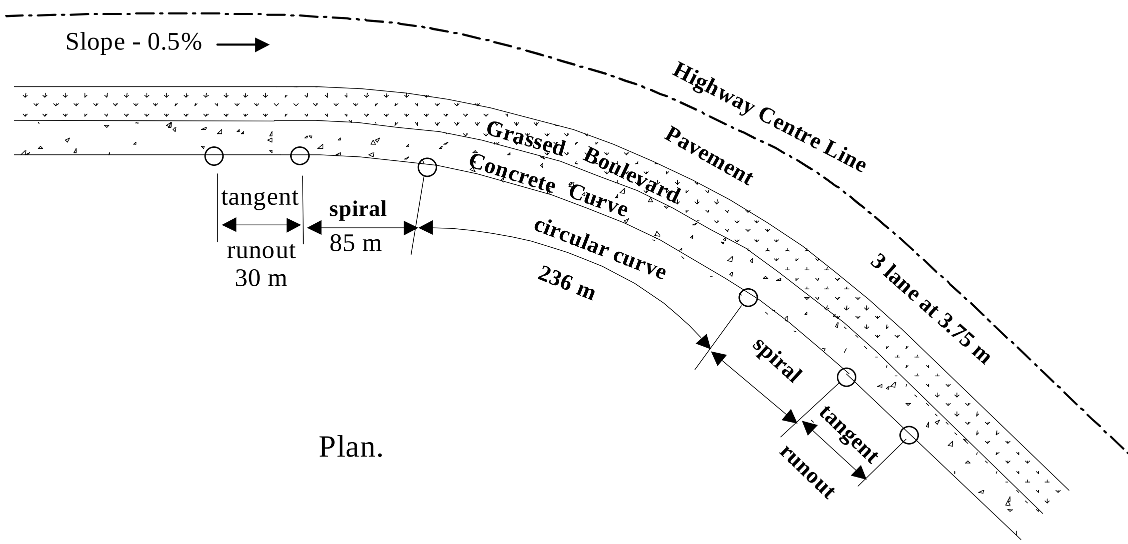
Preliminary Discussion

The change in S_x to create superelevation is determined with consideration for the safety of the travelling vehicle, the comfort of the rider and the need to drain the pavement effectively. The design criteria are normally specified in the highway geometric design manual (See Ontario Ministry of Transportation, 1985) and the design data provided by the road designer.

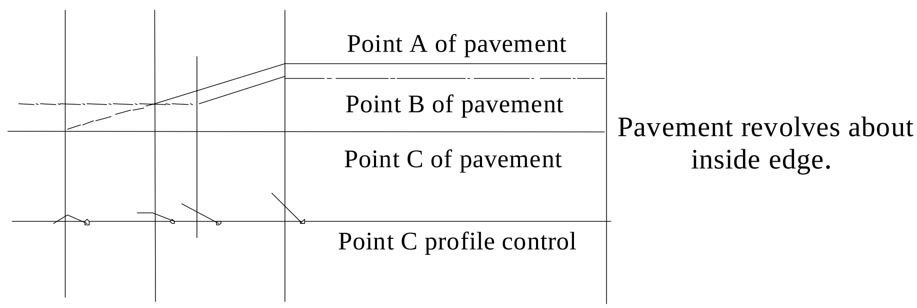
It is advisable to keep Q_c as low as possible at the upstream starting point of the horizontal curve. This can be done by providing an inlet at that point.

This design example will use MTO CBSpace to do the calculation. If designers have not used MTO CBSpace before, they are advised to go through Design Example 4.6 first before working on this design example. The procedure for using MTO CBSpace will not be explained again in this design example.

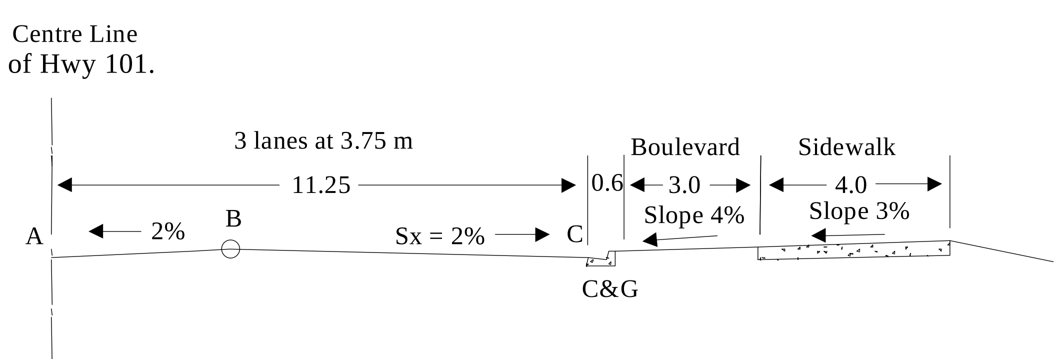
Figure (a): Site of Design Example 4.7



Plan.



Pavement Superelevation



Typical Half Cross-section.

Method

1. Enter the project data in the Project Panel. See Table (a).

2. Select the User-fixed spacing option from the MenuPage. This option gives the designer flexibility to decide where to place inlets, such as at critical locations in a superelevation.

3. On Line 1, enter the starting station **3150.0** in Col. 2. The cursor jumps to Line 2. Move it back to Line 1 and enter **0.0100** in Col. 19 for the carry-over runoff, Q_c.

4. Move the cursor to Line 2. Enter data in Col. 3 to 12. Enter **30.0** in Col. 16 for the first inlet on the curve. MTO CBSpace will calculate the remaining columns. Enter Col. 20 or simply press ENTER. MTO CBSpace will calculate the value (3180.0) for the station in Col. 2.

   Note: It is desirable to place an inlet at the downstream end of the tangent runout where the cross fall of the median lane becomes zero.

5. Repeat Step 4 for Line 3. Use $S_x = 0.04$. Try L = 85.

   Note:
   - X S_x is estimated as follows:
     - Avg. S_x of median lane = (0 + 0.06)/2 = 0.03
     - Avg. S_x of other lanes = (0.02 + 0.06) = 0.04
     - Avg. S_x of three lanes = (0.03 * 3.75 + 0.04 * 7.5)/11.25 = 0.036, say 0.04.
   - X There is no reason why an inlet must be placed at the downstream end of the transition curve. By trying L smaller and greater than 85, however, the designer will find that L = 85 is a reasonable choice. L < 85 uses too little spread and the spacing is not economical. L > 85 results in too much Q_c which is undesirable.

6. Repeat Step 4 for Line 4. Try L = 150 (allowable maximum spacing).
   Note: $Q_c = 0.0629$ is about twice Q_i. This large Q_c is undesirable becuase it increases the chance of pavement flooding if a downstream inlet is blocked. Normally, the designer can re-enter another trial value for L on the same line and observe the results. However, the change is made on Line 7 instead of Line 4 in order to preserve the results on Line 4 for comparison.

7. Move the cursor to Line 6 and enter **3265.0** in Col. 2 and **0.0214** in Col. 19 to provide MTO CBSpace with the necessary background data before going to Step 8.

8. Repeat Step 4 on Line 7. Try L = 75.

9. Repeat Step 4 on subsequent lines. Line 11 is the downstream end of the spiral curve (30 + 85 + 75 + 75 + 86 + 40 + 45 = 411).

Table (a): Sewer Inlet Spacing - Design Example 4.7

(To goto MENUPAGE, move cursor to green cell, press ENTER)

Sewer Inlet Spacing Calculation

Storm Frequency (Yr) ____________ ______
Rainfall Station ____________ ____
Rainfall Intensity (mm/h) 110
Allowable Spread (m) 1.9
Allow. Max. Spacing (m) 150
Curb & Gutter Type OPSD 600.01
Inlet Type OPSD 400.01
Inlet Time (min) 15

Hwy No.____________________________
W.P. No.
Designed By ____________ Date ____________
Checked By ____________ Date ____________

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
		Inpyt Data										Calc.Data	Design Results						
		Area Widths, W (m)				Runoff Coef., C				Gutter Grade So (m/m)	Cross-fall Sx (m/m)	Com-posite C	Gutter Flow Qg (m3/s)	Local Runoff Qr (m3/s)	Inlet Spacing L (m)	Calc. Spread T (m)	Inlet Capacity Qi (m3/s)	Carry Over Qc (m3/s)	
		W1	W2	W3	W4	C1	C2	C3	C4										
1	3150.0																	0.0100	
2	3180.0	7.50	0.60	3.00	4.00	0.95	0.95	0.60	0.95	0.05	0.02	0.89	0.0223	0.0123	30.0	1.1	0.0162	0.0061	
3	3265.0	11.25	0.60	3.00	4.00	0.95	0.95	0.60	0.95	0.05	0.04	0.89	0.0502	0.0441	85.0	1.1	0.0289	0.0214	
4	3415.0	11.25	0.60	3.00	4.00	0.95	0.95	0.60	0.95	0.05	0.06	0.89	0.0992	0.0779	150.0	1.1	0.0363	0.0629	
5																			
6	3265.0																	0.0214	
7	3340.0	11.25	0.60	3.00	4.00	0.95	0.95	0.60	0.95	0.05	0.06	0.89	0.0603	0.0389	75.0	1.1	0.0363	0.0240	
8	3415.0	11.25	0.60	3.00	4.00	0.95	0.95	0.60	0.95	0.05	0.06	0.89	0.0629	0.0389	75.0	1.1	0.0363	0.0266	
9	3501.0	11.25	0.60	3.00	4.00	0.95	0.95	0.60	0.95	0.05	0.06	0.89	0.0713	0.0447	86.0	1.1	0.0363	0.0349	
10	3541.0	11.25	0.60	3.00	4.00	0.95	0.95	0.60	0.95	0.05	0.04	0.89	0.0557	0.0208	40.0	1.1	0.0289	0.0268	
11	3586.0	11.25	0.60	3.00	4.00	0.95	0.95	0.60	0.95	0.05	0.04	0.89	0.0502	0.0234	45.0	1.1	0.0289	0.0213	
12																			
13																			
14																			
15																			
16																			
17																			
18																			
19																			
20																			

Design Example 4.8
Inlet Spacing Design, Road Sag

Required

Design inlet spacing on the sag of a highway.

Given

- X The plan, profile and cross section of highway as shown in Figure (a).
- X The highway grades, S_o = 2% on both sides of the road sag.
- X There is no additional flow from areas outside the roadway.
- X Design for the 25-yr storm. Rain intensity = 215 mm/h.
- X Cross fall, S_x = 0.02 m/m
- X Road sag length: 50 m on one side; 60 m on other side.
- X Type of curb and gutter: OPSD 600.01.
- X Type of combination inlet: OPSD 400.08
- X Maximum allowable spread, T = 1.5 m

Preliminary Discussion

The entire flow into the sag from the roadway should be drained by the inlet(s) to avoid ponding and its negative consequences. No carry-over runoff should be allowed.

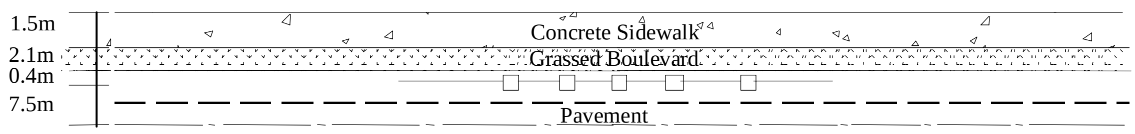
It is good practice to provide inlets at both upstream ends of the sag to capture as much of the upstream flow as possible, to prevent this flow from draining into the sag.

A safety factor of two is usually used in determining the required capacity of the inlets on a sag, to allow for blockage by debris. Twin inlets are sometimes used to provide the required capacity.

Combination inlets (OPSD 400.08) are used in this design example. The advantage of using combination inlets on a road sag is to help the inlets remain operative when the grates are blocked.

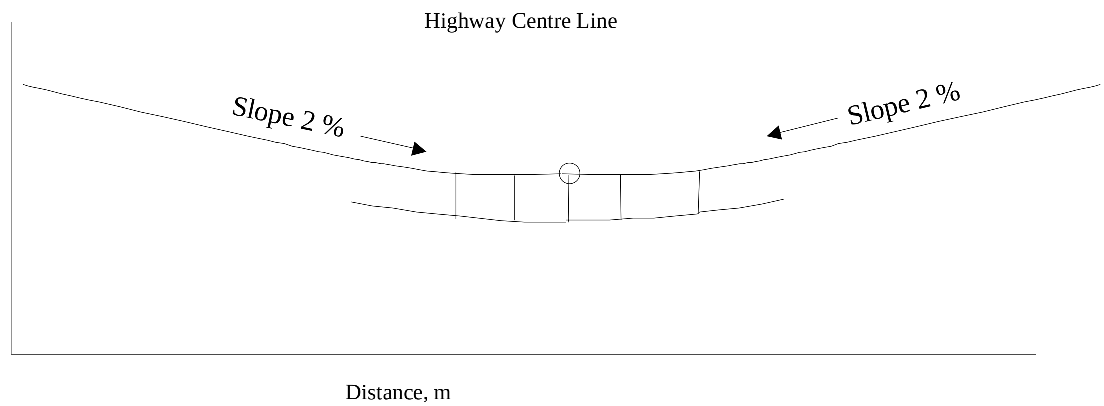
OPSD 400.08 is substantially similar to the City of Scarborough inlet Type C-93 tested by Marsalek (Marsalek, 1982). He found that the hydraulic performance of C-93 on a road sag is practically the same as inlets OPSD 400.01 and 400.03. For this design example, the hydraulic capacity of OPSD 400.08 is equated to that of OPSD 400.01.

Figure (a): Site of Design Example 4.8

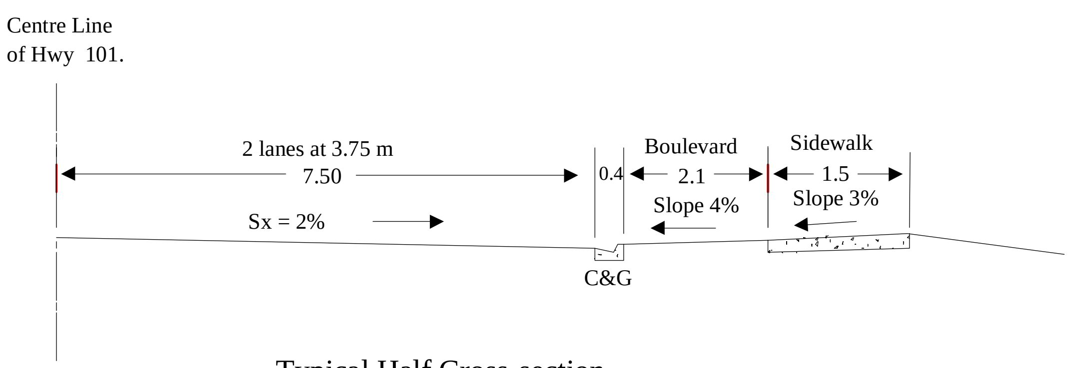


Centre line of highway and concrete median.

Plan



Profile.



Typical Half Cross-section.

Method

1. Composite runoff coeff., C = $(0.95 * (7.5 + 0.4 + 1.5) + 2.1 * 0.4) / 11.50$
   = 0.85 (8.10)

2. Q_r = $0.0028 * 0.85 * 215 * 11.5 * (50 + 60) / 10000$
   = 0.065 m^3/s (8.19)

3. carry-over flow from approach segments (assumed previously calculated)

   Q_{c1} = 0.015 m^3/s
   Q_{c2} = 0.020 m^3/s

   Total sag inflow, Q_g = 0.065 + 0.015 + 0.020 = 0.10 m^3/s

4. For T = 1.5 m and S_x = 0.02 m/m
   Capacity of single OPSD 400.01= 0.038 m^3/s (Design Chart 4.14)
   No. of combination inlets required = 0.10 / 0.038 = 2.63, say 3
   Capacity of 3 inlets = 3 x 0.038 = 0.114 m^3/s

5. Place one inlet at the bottom of the sag and one on each side of the sag at a vertical distance of 0.06 m from the bottom of the sag as recommended by the hydraulic tests (Marsalek, 1982).

Design Example 4.9
Roadside Ditch Inlet Spacing Design

Required

Determine the spacing of inlets located in a median ditch of a highway. The ditch collects runoff of minor storms from the roadway and drains into a storm sewer system.

Given

- A 22 m wide median ditch drains two 3.75 m highway lanes on each side of the ditch.
- Design for the 10-year storm. Rain intensity = 110 mm/h.
- The ditch has a trapezoidal cross section. Ditch side slope, S_s = 6:1. See Figure (a).
- Type of ditch inlet: OPSD 403.01.
- Allowable maximum spacing = 150 m.

Preliminary Discussion

Ditch inlet spacing design is very similar to pavement inlet spacing design (see Design Example 4.6). One difference is that a different type of inlet is used. In addition, the inlet spacing should ensure that the elevation of maximum water surface in the ditch is not higher than that of pavement subdrains as discussed in the Roadside Ditch section of this chapter.

Method

1. Composite runoff coefficient, C

   $= (2 * 7.5 * 0.95 + 22 * 0.40) / 37 = 0.62$

2. Try allowable maximum inlet spacing.

   $Q_r = 0.0028 * 0.62 * 110 * 37 * 150 / 10000 = 0.105\ m^3/s$ (8.19)

3. Allow for 50% reduction in capacity due to blockage.

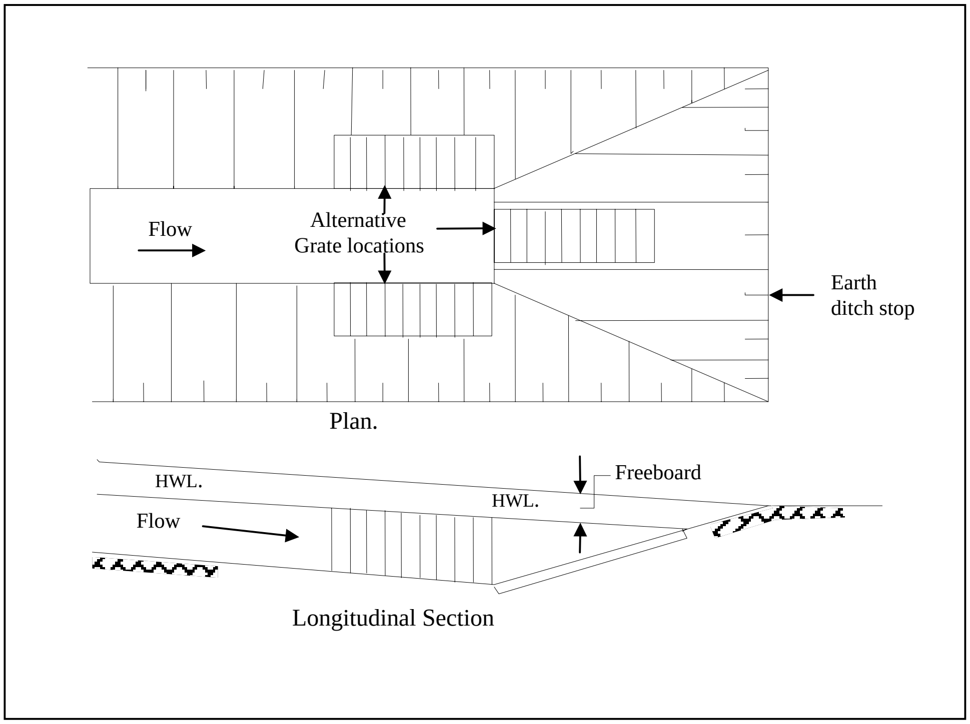
   Required inlet capacity, Q_i $= 2.0 \times 0.105$
   $= 0.210\ m^3/s$

4. Determine the water depth in the ditch required to provide an inlet capacity of $0.210\ m^3/s$.

The width of OPSD 403.01 = 0.762 m
The equivalent required capacity of the inlet per m = 0.210 / 0.762 = 0.28 m^3/s
The water depth for the required inlet capacity = 0.16 m (Design Chart 4.20)

Note: Check if the pavement subdrains will be submerged at the required water depth in the ditch. If they will, reduce the inlet spacing and/or revise the design of the median ditch.

Figure (a): Placement of Inlet in Ditch



Bridge Deck Drainage*

The Function of Bridge Deck Drainage

Bridge deck drainage may be considered as a special case of pavement drainage. Bridge deck drainage is important to traffic safety in wet weather and in winter.

Deck Inlets

From Design Example 4.6 for pavement drainage, it can be concluded that deck inlets may not be needed for many short bridges if adequate gutter inlets are provided immediately upstream of the deck to intercept the runoff before it reaches the bridge. Also, avoid locating a bridge on zero longitudinal grades and vertical sags. The minimum grade for a bridge deck to be hydraulically functional is 0.5%. (U.S. Federal Highway Administration, 1984).

If deck inlets are needed, use the design methods provided in Design Examples 4.6 and 6.7, but use deck inlets instead of ordinary gutter inlets. Figure 4.5 shows the deck inlet types most commonly used by MTO. Their hydraulic capacities are provided in Design Chart 4.21.

Downpipes

A downpipe conveys the runoff from a bridge deck inlet to a storm sewer or ditch at ground level. Figure 4.6 illustrates a downpipe arrangement. A downpipe for a bridge serves similar purposes as does a downpipe for a large office building, that is, to provide good drainage and to give a good visual appearance. For drainage design purposes, a downpipe for bridge deck drainage should:

- Have a hopper attached to its upper end to guide the turbulent flow from the inlet into the downpipe. The hopper should be deep and large enough to contain the runoff at its design rate plus ample allowance for flow turbulence.
- Have a cross section larger than the spout of the tributary bridge deck inlet.
- Allow an appropriate space between the spout of the deck inlet and the sides of the hopper to permit entrained air to escape into the atmosphere and not forced into the downpipe.
- Have a cleaning eye at every bend of the downpipe less than 135^{o}.
- Have no projecting objects inside the downpipe that may catch debris in the flow.
- Be leak-proof and suitable for installation on the external faces of the structure in all seasons.

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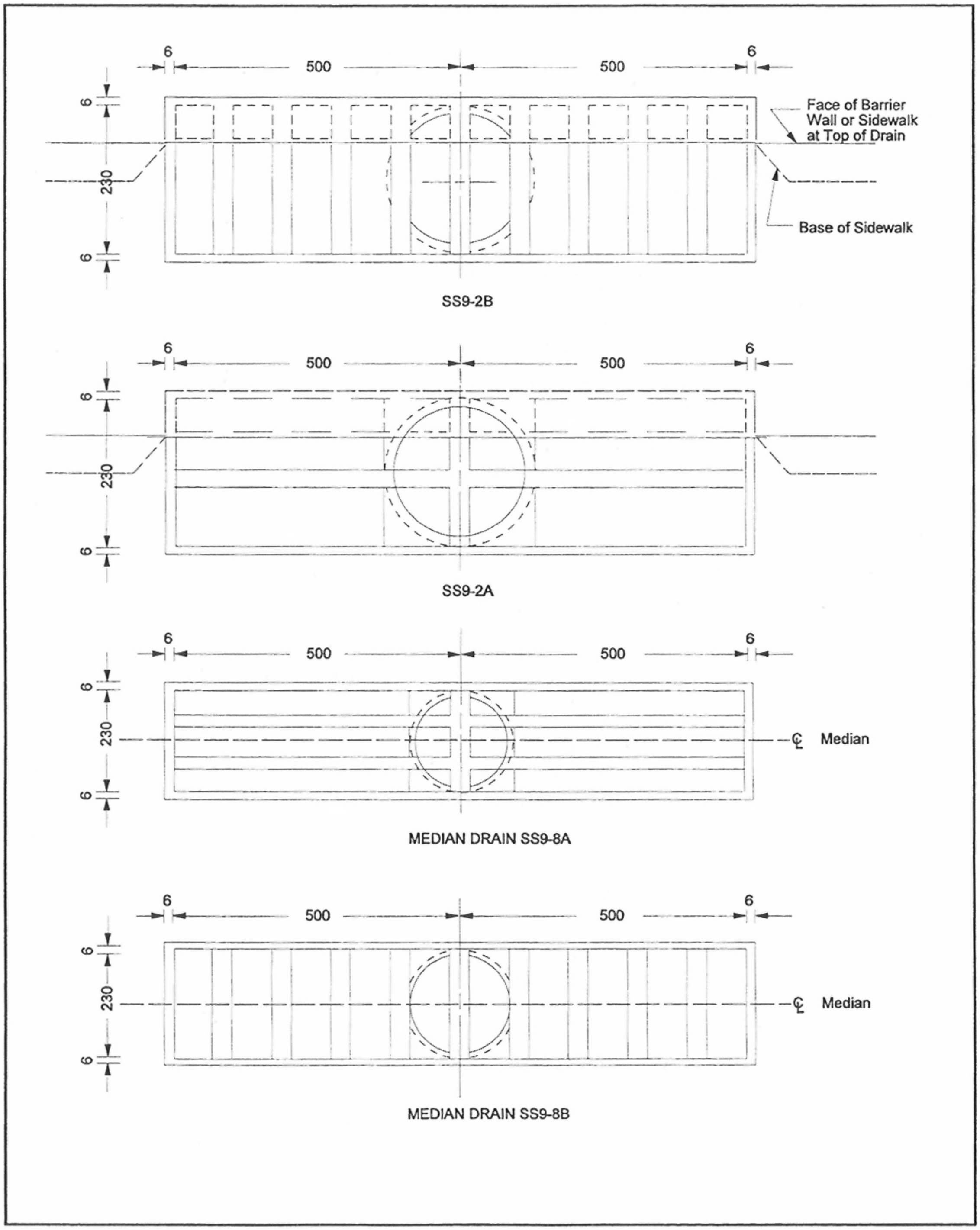
* This section is a continuation of Step 5 of the Detail Design Process.

Figure 4.5: Bridge Desk Drainage Inlets



Free-dropping of Runoff

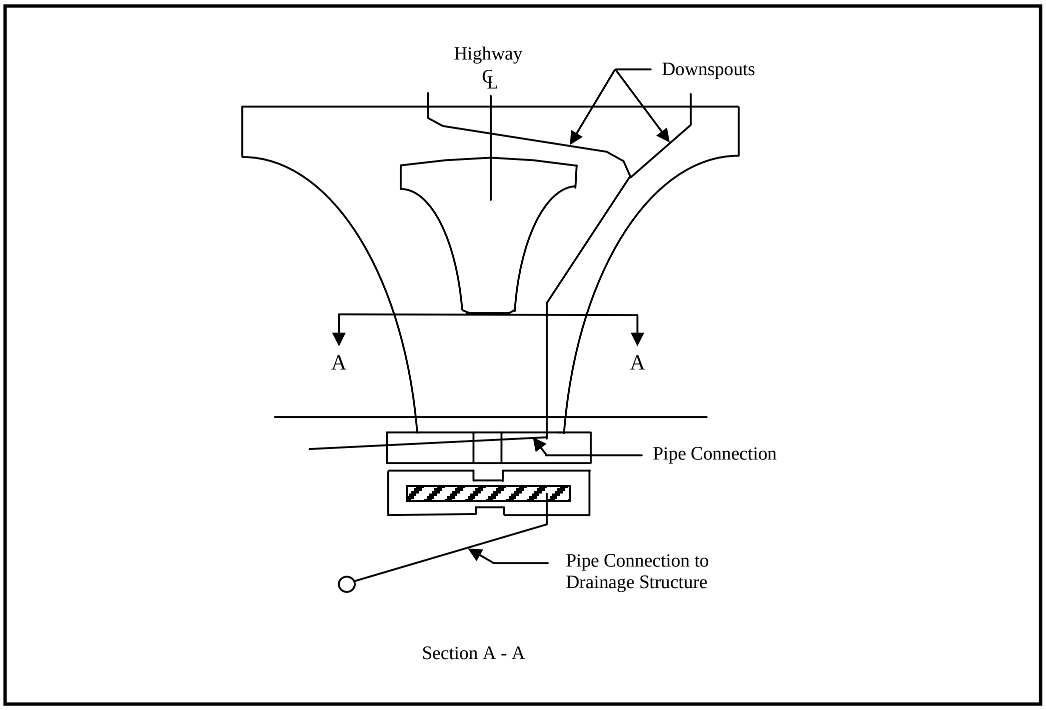
It is not good practice to allow runoff to drop from the spouts of deck inlets to the watercourse or land under the bridge. The environmental and aesthetic impact associated with this practice cannot justify any cost savings that might be achieved. The runoff should be brought down to the drainage system at ground level with downpipes.

Ground Level Drainage Outlets

It is good practice to avoid discharging the flow from a downpipe directly onto the ground surface or the watercourse. Ideally, it should be connected to a storm sewer or ditch. If a storm sewer or ditch is not available, consider providing a grassed swale to intercept the downpipe and let the swale run for a good distance before it discharges to a watercourse. Some erosion protection for the swale with riprap may be needed around the lower end of the downpipe.

In some special situations involving sensitive streams, some storm water quality improvement for this runoff may be required. This requirement should normally be determined at the preliminary design stage.

Figure 4.6: Bridge Deck Drainage Downpipe Arrangement



Wet Ponds/Extended Dry Ponds*

Wet Pond Features at a Glance

Wet ponds are one kind of facilities for stormwater quality management. For a discussion of preliminary design considerations for stormwater quality management, refer to Chapter 3.

The following sections discuss the various elements of a wet pond. The discussion refers to the figures in Design Example 4.11 for illustration.

Flow Splitting

During a storm, the runoff from the contributing drainage areas flows toward the pond via the storm sewer(s) or ditch(es). At any moment during the storm, when the runoff arrives at the flow splitter, the splitter handles the runoff according to one of the following situations that is prevailing:

Situation 1: Pond Not Full. All the runoff enters the pond. No overflow at the splitter.

Situation 2: Pond Full. Runoff Rate Not Greater than Flow Splitter Threshold Rate. All the runoff enters the pond. No overflow at the flow splitter. The *threshold rate* is the maximum flow rate specified in the design that is allowed to enter the pond when the pond is full.

Situation 3: Pond Full. Runoff Rate Greater than Flow Splitter Threshold Rate. The fraction of the runoff less than the splitter threshold rate enters the pond. The remaining fraction bypasses the pond and overflows to the receiving stream or drain directly to a water quantity control facility, if one is required and provided.

The Uniform Flow Distributor

The uniform flow distributor is used for distributing the inflow uniformly across the width of the main body of the pond. Uniform flow distribution is an important assumption made in the estimation of the pond solids removal performance. Short-circuiting occurs if the flow is not uniformly distributed, that is, the flow takes the shortest route to the outlet, within a narrow section of the pond width. Short-circuiting under-utilizes the storage capacity.

* This section is a continuation of Step 5 of the Detail Design Process.

The Forebay

The forebay is a storage area located immediately downstream of the inlet and upstream of the main body of the pond. The forebay is to be designed such that the flow enters the main body of the pond over the entire width of the forebay. The volume of the forebay and the main body of the wet pond make up the total storage space. Although many practitioners consider the forebay to be useful in pre-settling some contaminants borne by the incoming runoff, the effectiveness of pre-settling is yet to be verified. Although the forebay may provide some benefit in settling large particles, it may be prudent in a design to regard the forebay more as a uniform flow distributor.

The Permanent and Active Pools

The main body consists of two pools: a permanent pool at the bottom and an active pool on top of it. The water elevation in the permanent pool is the lowest level to which the pond can be drawn down to during normal operation. The water in the permanent pool is not drained. It mixes with incoming stormwater and dilutes it. The water quality in the permanent pool is considered to be good due to the extended detention period provided (equal to the interval between storms). The water volume in the active pool varies according to inflow conditions. This main body of the pond accounts for the solid settling capacity of the pond.

The Outlet

The outlet of the wet pond may be a sloped pipe or a perforated riser. It draws water from below the water level of the permanent pool in the pond. It controls the release rate of the active pool of the pond. For a pipe flow, control is provided through the placement and sizing of an orifice plate or through a sluice gate control. For a perforated riser, flow control is provided through the appropriate sizing of the perforations. The outlet design rate is set equal to the threshold rate of the flow splitter.

The Emergency Bypass

The emergency bypass provides a safety outlet for high flows that may enter the pond unexpectedly. It is not expected to be used under normal operating conditions.

Design Method

Prior to proceeding to detail design analysis, it may be necessary to refine the layout plan of the proposed wet pond, developed in the preliminary design, and organize all the available data.

Refine the Layout Plan

Here are possible points to consider in refining the layout plan.

- Is the site area sufficient to accommodate the pond and the maintenance access?
- What site grading will be required?
- Will the soil be suitable and will there be groundwater problems (check with geotechnical professionals)?
- If the site is on a flood plain, will high water flood the pond and will the pond be able to discharge during a high water period?
- Is there any need to revise the positioning of the various elements of the pond? For example, is the outlet location suitable?
- What landscaping opportunities will be available (work with the landscape designers)?

Organize the Data

The following are possible action items to consider.

- Organize the drainage map data and precipitation data into the formats required by the computation methods.
- Do any preliminary data analysis as necessary. For example, if the computation method to be used is a derived probability distribution method such as the MTO SUDS extension model, the precipitation data will have to be synthesized into specific statistical parameters, as described in Design Example 4.10.
- Review the design criteria and preliminary design results to see if there is any need for clarification.

Do Design Analysis

The application of computation methods is best illustrated by design examples. Two design examples are presented. Design Example 4.10 shows a preliminary analysis of a wet pond using the MTO SUDS Extension model. Design Example 4.11 shows a detail hydraulic design of a wet pond using the MTO SWMM Extension model in the continuous simulation mode.

The *expected performance* referred to is based on computational analysis and not on field monitoring of a pond in operation. In analysing the expected performance of a stormwater quality facility, it is essential to use long-term data (say more than 10 years of continuous records). The computation methods used in the two design examples are based on this principle. If single storm event data are used instead, the results will be misleading. For a discussion of the underlying principle of this data consideration and the computation methods, see Chapter 8.

Extended Dry Ponds

An extended dry pond is essentially the same as a wet pond without a permanent pool. The design method discussed for wet ponds is generally applicable to the design of extended dry ponds.

Design Example 4.10
Preliminary Analysis of Wet Pond

Required

Do a preliminary analysis of alternative sizes of a proposed wet pond. The results are intended to be used in selecting a preferred size for further study. Use the MTO SUDS Extension model.

Given

- Drainage areas of the pond. The data include the boundaries of the drainage areas, relevant land use and soil properties. The data, after synthesis, are:
  - Catchment area = 100 ha
  - Runoff coef. = 0.5
  - Avg. depression storage = 3.0 mm.
- AES longest available hourly precipitation records of Pearson International Airport synthesized into statistics. See Table (a).
- Tentative pond dimensions based on site conditions and guided by information on drainage areas:
  - Maximum pond storage surface area = 20,000 m^2
  - Maximum pond storage total depth = 2.5 m
- Particle size distribution of contaminants in runoff as shown in Table (b).
- Design criteria
  - Average detention time = 12 h
  - Minimum avg. removal efficiency = 75%.

Preliminary Discussion

- In order to be able to do wet pond analysis properly, the practitioner will have advanced knowledge of hydrology, hydraulics and drainage management practice. He or she will also have extensive experiences in hydraulic design of advanced stormwater management facilities.
- This analysis incorporates three fundamental principles:

  1. The particle size distribution of the contaminants in the runoff are site-specific. The method will allow site-specific data to be used.

  2. All surface runoff from the drainage areas of the pond in every storm event of the year will flow to the pond (i.e. integration of stormwater quantity and quality management). The flow will be split on arrival at the inlet of the pond as discussed in the main text.

3. The expected contaminant removal efficiency of the pond is as defined in the main text.

For a discussion of the underlying reasons for adopting these fundamental principles, see Chapter 8.

- The advantage of MTO SUDS Extension model is that it is easy to use and its data requirements are very modest. As well, it is built on theories that support the above three principles. For a discussion of the model, see Adams, 1996. A brief summary of this method is presented in Chapter 8. See also the README.TXT file accompanying the model for user instructions.

Table (a): Pearson Airport Rainfall Statistics

Avg. annual rainfall = 550.8 mm	Avg. event rainfall = 5.1 mm	Avg. event duration = 3.6 h	Avg. interevent time = 80 h

Table (b): Particle Size Distribution of Contaminants

Size Fraction	Particle Size (μm)	% of Mass	Settling Vel. v_s	
1	20	20	0.00000254 m/s	= 9.14 mm/h
2	40	10	0.00001300	= 46.8
3	60	10	0.00002540	= 91.4
4	130	20	0.00012700	= 457.2
5	400	20	0.00059267	= 2133.6
6	4000	20	0.00550333	= 19812

Method

The input data to MTO SUDS Extension model are shown in Table (c). Output results for various trial active storage volumes are shown in Table (d). The results in Table (d) are all based on a ratio of depth of active pool to depth of permanent pool of 1:2.

Run the model again with the same given data (Table (c)) but with different storage volumes, different depths and different ratios of depth of active pool to depth of permanent pool as indicated in Table (e). The results of contaminant removal efficiencies for these runs are also indicated in Table (e).

Table (c): Input Data to MTO SUDS Extension Model

ζ	= 1/avg. event volume	= 1/5.0	= 0.200 1/mm
λ	= 1/avg. event duration	= 1/3.55	= 0.282 h
Ψ	= 1/avg. interevent time	= 1/43.5	= 0.023 1/h
Φ	= Runoff coefficient	= 0.5	
S_d	= Depression storage	= 3.0 mm	
A_c	= Catchment area	= 100 ha	
n	= Turbulence factor	= 3	
h_a	= Active storage depth	= 0.5 m	
h_p	= Permanent storage depth	= 1.0 m	
t_d	= Detention time	= 12 h	

Particle size distribution

Size #	Mass %	Set.Vel. (mm/h)
1	20	9.14
2	10	46.8
3	10	91.4
4	20	457.2
5	20	2133.6
6	20	19812

Table (d): Results of MTO SUDS Extension Model

Active Pond Vol., V_a (m^3)	Permanent Pond Vol.,V_p (m^3)	Removal Efficiency %, C_p
2500	5000	78.63
3000	6000	80.67
5000	10000	83.10
7000	14000	83.38
9000	18000	83.41

Table (e): Summary of Results for Different Pond Sizes

h_a (m)	h_p (m)	h_a:h_p	V_a (m^3)	V_p (m^3)	V_a+V_p (m^3)	C_p %
0.50	1.0	1:2	7000	14000	21000	83.38
0.50	1.0	1:2	9000	18000	27000	83.41
0.75	1.50	1:2	7000	14000	21000	79.61
0.75	1.50	1:2	9000	18000	27000	79.64
0.60	0.90	2:3	7000	10500	17500	84.18
0.60	0.90	2:3	9000	13500	22500	84.33
0.90	1.35	2:3	7000	10500	17500	80.45
0.90	1.35	2:3	9000	13500	22500	80.59

Discussion of Results

- Table (d) shows that a pond of volume 7,500 m^3 (2,500 + 5,000) will satisfy the removal efficiency requirement. In reality, however, MTO SUDS Extension model tends to over estimate a pond's performance by 10-20% (Adams, 1996). Therefore, a likely volume to start a more detailed study will be about 21,000 m^3 (7,000 + 14,000).
- Table (d) also shows that once the removal efficiency has reached about 83%, there is very little benefit for increasing the pond volume. Corresponding analysis using a continuous simulation model shows that the plateau of the removal efficiency vs volume curve will occur at a larger volume than predicted by this model (Adams, 1996).
- Table (e) shows that, for the same pond volume, say V_a+V_p = 21,000, and the same h_a:h_p ratio (1:2), the removal efficiency is better for a shallower pond. This is expected.
- Table (e) also shows that an h_a:h_p ratio of 1:2 is not necessarily the best ratio. As shown in this table, the 2:3 ratio outperforms the 1:2 ratio. What ratio will be the best, however, cannot be predicted generally.
- In the light of the information in Tables (d) and (e), it is advisable to try a variety of storage volumes, depths and ratios of depths to determine which combination will be the most preferred choice in a given study. MTO SUDS Extension model can be a useful tool for this purpose.

Design Example 4.11
Detail Hydraulic Design of Wet Pond

Required

A detail hydraulic design of a wet pond for stormwater management, including its inlet, outlet and emergency bypass. Use SWMM and MTO SWMM Extension models.

Given

The same data and design criteria as given in Design Example 4.10, with the following changes based on the findings of a preliminary design report :

- A plan and typical sections of the pond adjusted for site conditions. See Figures (a) and (b).
- Regulatory flow, Q_{reg} = 7.0 m^3/s
- Active storage depth = 0.5 m; volume = 9,000 m^3
- Permanent storage depth = 1.0 m; volume = 18,000 m^3/s
- Max. pond inflow rate = Q_{25} = 5.0 m^3/s
- Average detention time = 12 h
- Grass-lined approach channel: Invert slope = 0.001 m/m; side slope = 2:1

Preliminary Discussion

In this design example, the principle of estimating contaminant removal and the principle of flow splitting when the pond is full are as discussed in Design Example 4.10. When the pond is not full, however, the rate of maximum runoff permitted to enter the pond is set at the peak flow rate of the 25-year storm, Q_{25}, instead of the regulatory flow, Q_{reg}. Q_{25} is considered high enough in which the runoff will be quite "clean", so there is no appreciable benefit to tax the capacity of the pond to admit a flow higher than Q_{25}. Furthermore, using Q_{25} instead of Q_{reg} to design the pond inlet will help avoid a large hydraulic head required to drive Q_{reg} through the inlet.

This design example assumes that major design considerations have been addressed in the preliminary design stage. Among these considerations are:

- the justification for providing a wet pond, the preliminary pond size, and design criteria;
- the location of the pond and site feasibility;
- maximizing the pond length-to-width ratio to reduce possible flow short-circuiting; and
- fitting the pond into the natural setting of the site and landscape design of the site.

Figure (a): Wet Pond Layout

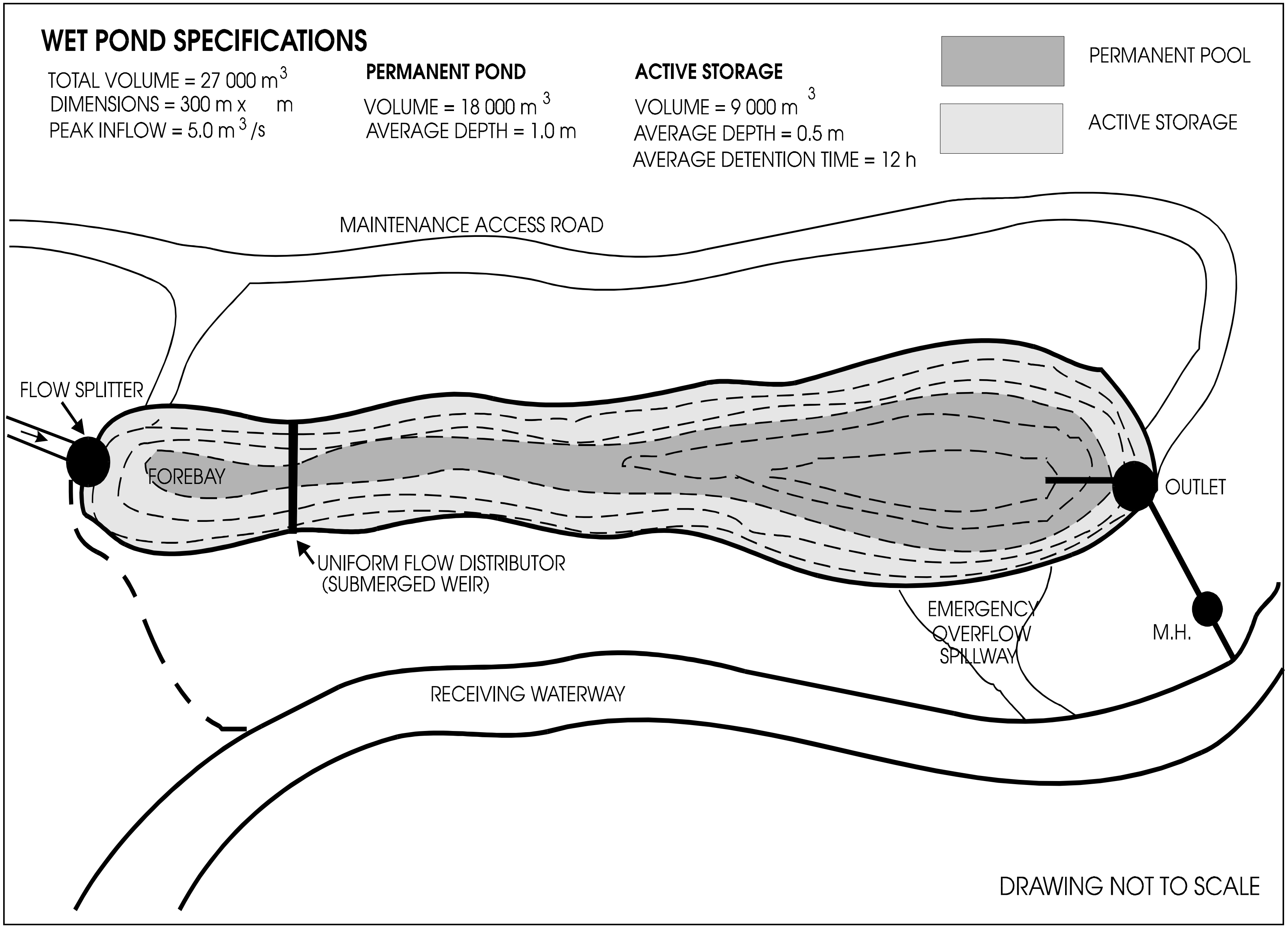
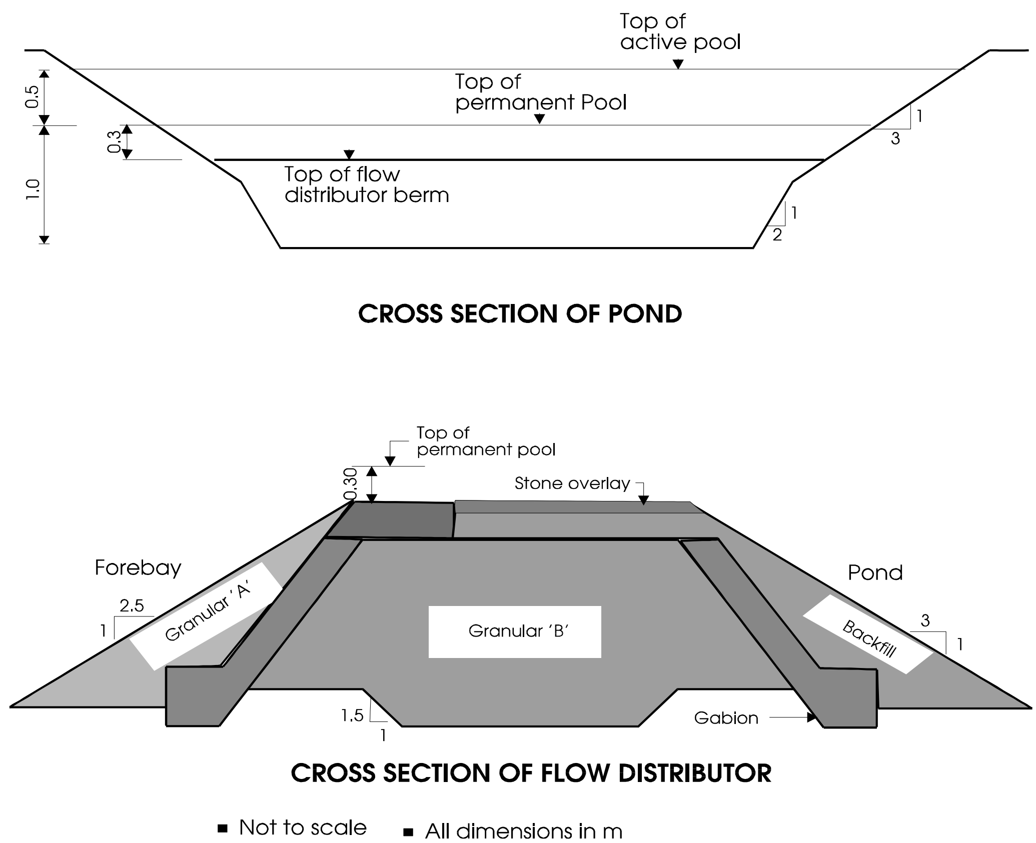


Figure (b): Typical Sections of Wet Pond



Pond Analysis

This design example uses SWMM and MTO SWMM Extension models in the continuous simulation mode to perform the analysis. In order to be able to do this design task successfully, the practitioner will need extensive knowledge of, and experiences in, advanced hydrologic modelling.

A summary discussion of MTO SWMM Extension model and SWMM is provided in Chapter 8. For a more detailed discussion of the MTO SWMM Extension model, see Adams, 1996. See also the README.TXT file accompanying MTO SWMM Extension model for user instructions. User instructions for SWMM may be found in that model's user manual.

Step 1: Run SWMM Model (RUNOFF Block)

Compile catchment data required by RUNOFF Block. The compiled data are shown in Table (a).

Table (a): Compiled Catchment Data

Area	100 ha
Imperviousness	50%
Slope	1%
Impervious Manning's n	0.013
Pervious Manning's n	0.25
Impervious depression storage	1.5 mm
Pervious depression storage	4.5 mm
Minimum infiltration rate	7.5 mm/h
Maximum infiltration rate	50 mm/h
Infiltration decay rate (Horton's equation)	0.00055 s^{-1}
Catchment width	2000 m

Obtain the most applicable long-term monthly evaporation rates. In this design example, the AES data from Hamilton are selected as shown below (rates in mm):

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
0.0	0.0	0.0	87.9	111.0	124.8	143.5	123.3	81.3	46.2	0.0	0.0

Use the given and collected data to prepare an input file for the RUNOFF Block. The AES records contain 33 years of data for the months from April to October each year. Run SWMM and save a digital file of the entire output. The results are summarized in Table (b).

Step 2: Run MTO SWMM Extension Model

Table (c) shows the input for running the model. The model accepts input in a question-and-answer mode at runtime. Note that the time step for the model run is 300 seconds and not 60 minutes (used by the AES rain data), because settling calculation is sensitive to large time steps. The SWMM RUNOFF Block output file, MTO100.inp, is used as input. The "first flush" pond means that not all the runoff will enter the pond. This flow control is achieved by the flow splitter at the pond inlet.

Table (b) SWMM RUNOFF Results

Description	Result
Catchment area	100 ha
Precipitation*	550.9 mm
Infiltration*	274.1 mm
Evapotranspiration*	42.3 mm
Surface runoff*	245.1 mm
Continuity error	1.9 %
Maximum runoff flow (Aug. 22, 1968)	7.9 m^2/s
* Avg. annual value	

Table (c): Input for MTO SWMM Extension Model

Input filename	mto100.inp
Detailed output filename (Optional)	test2.out
Summary output filename (Optional)	test2.sum
Short circuit factor (1 (worst) > 5 (best))	3
# of active storage to be simulated	1
Active storage volume, V_a (m^3)	9000
Active storage average depth, h_a (m)	0.5
# of permanent storage to be simulated	1
Permanent storage, V_p (m^3)	18000
Permanent storage average depth, h_p (m)	1.0
Data time step (min.)	60
Calculation time step (s)	300
Simulate summer period only? (Y or N)	y
Simulate "first flush" pond? (Y or N)	y
"First flush" pipe capacity (m^3/s)	5
Number of particle sizes to be simulated	6
Particle size #	1
Particle size (mm)	0.02
Percent by mass (%)	20
Settling velocity (m/s)	0.0000025
Particle size #	2
Particle size (mm)	0.04
Percent by mass (%)	10
Settling velocity (m/s)	0.0000130
Particle size #	3
Particle size (mm)	0.06
Percent by mass (%)	10
Settling velocity (m/s)	0.0000254
Particle size #	4
Particle size (mm)	0.13
Percent by mass (%)	20
Settling velocity (m/s)	0.0001270
Particle size #	5
Particle size (mm)	0.40
Percent by mass (%)	20
Settling velocity (m/s)	0.0005927
Particle size #	6
Particle size (mm)	4.00
Percent by mass (%)	20
Settling velocity (m/s)	0.0055033

Step 3: Review Modelling Results

The summary output is shown in Table (d). It can be seen that the proposed pond meets all the design criteria. Note that the results refer to the entire simulation period of 33 years. In particular, the number of overflow events in 33 years is 54, that is 54/33 or 1.6 overflow events per year.

Table (d): MTO SWMM Extension Model Results

Permanent storage: Volume = 18000 m^3 Depth = 1.0 m
Active storage: Volume = 9000 m^3 Depth = 0.5 m
Detention time = 12 h
Short circuit factor = 3

Continuous operation:
- Total inflow volume = 3679825 m^3
- Total overflow volume = 319996 m^3
- TSS in inflow = 551974 kg
- TSS removed = 419581 kg
- TSS in outflow = 87347 kg
- TSS in overflow = 47999 kg
- Number of overflow = 54
- % TSS removal = 75.5%

Sediment Summary
Sediment discharge - Not removed:

Particle Size (mm)	% Mass	Settling Vel. (m/s)	Outflow TSS (%)	Overflow TSS (%)	Total TSS (%)
0.020	20	.0000025	40.0	20.0	32.9
0.040	10	.0000130	7.8	10.0	8.6
0.060	10	.0000254	7.6	10.0	8.5
0.130	20	.0001270	14.9	20.0	16.7
0.400	20	.0005927	14.8	20.0	16.7
4.000	20	.0055033	14.8	20.0	16.7

Note that many alternative designs of the pond can be achieved by varying the following key design parameters: permanent storage volume and depth; active storage volume and depth; and threshold inlet rate (or retention time). Table (e) shows a summary of several runs of MTO SWMM Extension model with different pond volumes and depths for comparison. The other input data are unchanged in all the runs.

Table (e): Comparison of Different Pond Sizes

h_a	h_p	h_a:h_p	V_a	V_p	V_a+V_p	%TSS Removal	%TSS Overflow	No. of Overflow
0.5	1.0	1:2	8,000	16,000	24,000	73.0	10.8	69
0.5	1.0	1:2	9,000	18,000	27,000	75.5	8.7	54
0.5	1.0	1:2	11,000	22,000	33,000	79.2	6.0	30
1.0	1.5	2:3	12,000	16,000	28,000	75.5	5.0	26
0.5	1.5	1:3	9,000	18,000	27,000	74.7	8.7	54

There are several interesting points to note from Table (e):

- For the same h_a:h_p ratio, say 1:2, TSS removal increases steadily with increase in total pond volume. However, there is not sufficient information in the table to conclude where the TSS removal vs pond volume curve will reach a plateau.
- A comparison of row 2 and row 4 suggests that for about the same total volume, a 2:3 depth ratio yields less overflow events. Again, this is an observation from the table and cannot be considered as a generally applicable conclusion.
- A comparison of row 2 and row 5 suggests that when the permanent storage depth is large compared with the active storage, there is no advantage to further increase the permanent storage depth at the expense of the active storage depth.
- The information from the table suggests that it is advisable to run MTO SWMM Extension model for several alternative pond volumes and depths and select a preferred solution. One run of the model takes only a few minutes.

Hydraulic Design of Pond Elements

The design of pond elements in this design example is done for a pond with an active pool volume of 9,000 m^3 and depth of 0.5 m and a permanent pool volume of 18,000 m^3 and depth of 1.0 m.

Flow Splitter

The flow splitter is a hydraulic device designed to control the inlet flow rate (the threshold rate) of the pond. In this design example, the device consists of an orifice inlet and a side weir as shown in Figure (c). A possible alternative to the orifice may be a Parshall flume. A Parshall flume is normally used for flow measurements and its dimensions for various flow capacities have been pre-determined and calibrated in hydraulic laboratory tests. Therefore, a Parshall flume should offer the best accuracy in the control of the maximum permissible flow rate to the pond. However, the construction of a Parshall flume is more complex than that of an orifice. A very good, practical discussion of Parshall flume is available in Chow, 1959 and U.S. Department of Agriculture, 1950.

The hydraulic design of the flow splitter (Figure (c)) is presented below.

(a) Determine flow depths in approach channel

Channel invert slope, S = 0.001 m/m
25-yr design flow, Q_{25} = 5.0 m^3/s
Regulatory flow, Q_{reg} = 7.0 m^3/s
Manning's n for grass lining = 0.05 (Design Chart 2.01)

For Q_{25}:

Try flow depth = 1.6 m
Area of flow, A = (8.4 + 2.0) * 1.6 / 2 = 8.32 m^2
Wetted perim., P = 2.0 + 2 * 1.6 * $\sqrt{5}$ = 9.16 m
Using Manning equation,
$Q = A * (A / P)^{2/3} * (\sqrt{S}) / n$ = $8.32 * (8.32 / 9.16)^{2/3} * (\sqrt{0.001}) / 0.05$ (8.66)
= 4.94 $\approx$ 5.0 m^3/s O.K.

For Q_{reg} :

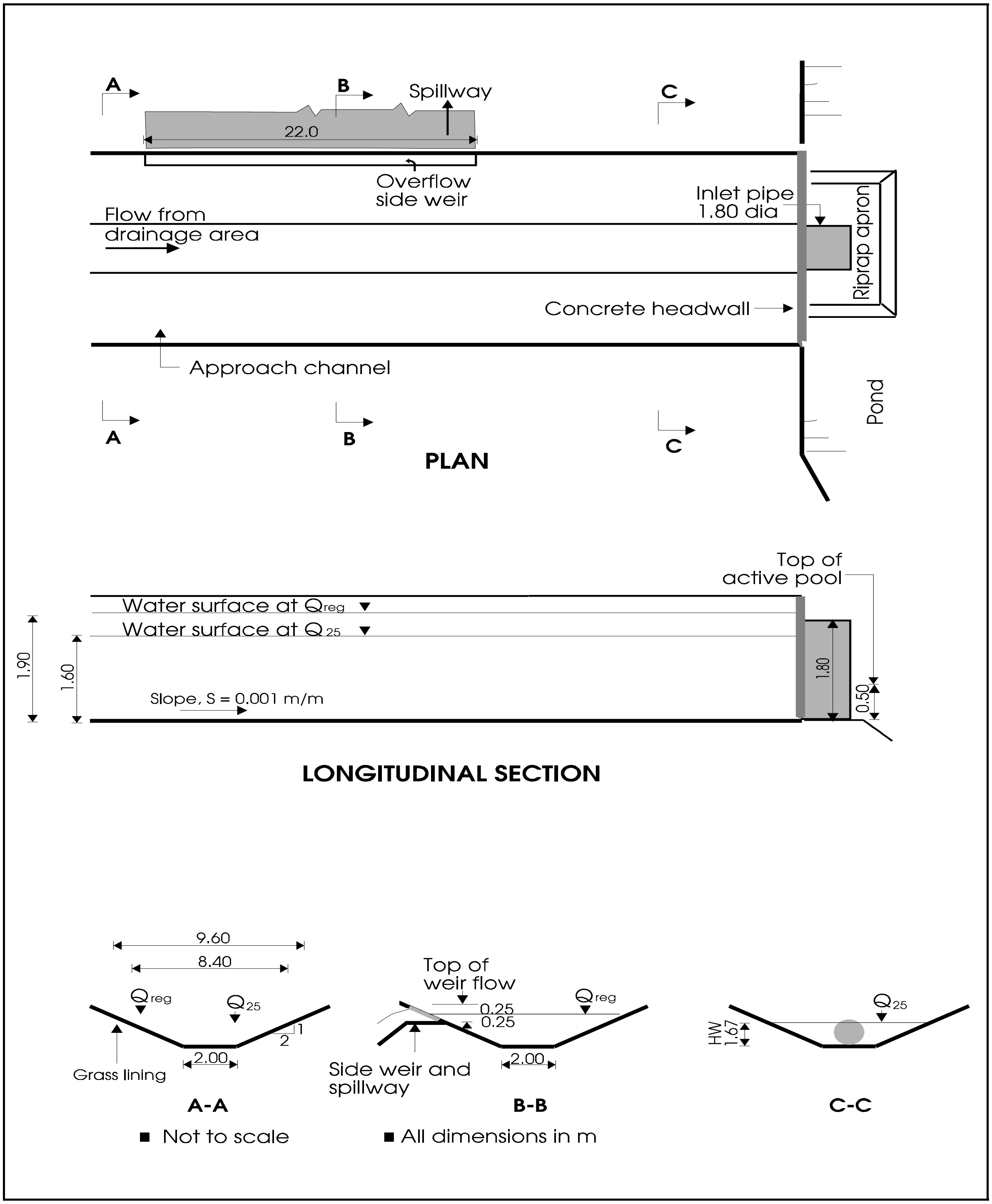
Try flow depth = 1.9 m
Area of flow, A = (9.6 + 2.0) * 1.9 / 2 = 11.02 m^2
Wetted perim., P = 2.0 + 2 * 1.9 * $\sqrt{5}$ = 10.50 m
$Q = 11.02 * (11.02 / 10.50)^{2/3} * (\sqrt{0.001}) / 0.05$ (8.66)
= 7.20 > 7.0 m^3/s O.K.

(b) Design the inlet to pond

The inlet to the pond is a concrete pipe designed to act as an orifice for more accurate control of the maximum permissible flow rate to the pond. The usual formula for a small orifice is not applicable because the required opening is large and the hydraulic head driving the flow through the orifice cannot be deemed to be constant from the crown to the invert of the orifice. Use Design Chart 2.31 for a circular pipe under inlet control which, in fact, acts as an orifice.

To ensure that the inlet pipe will actually flow under inlet control condition, the pipe must be short and the tailwater at the outlet of the pipe must not be higher than mid-height of the installed pipe.

Figure (c): Flow Splitter and Approach Channel



Use a pipe size large enough not to raise the water surface of the approach channel significantly, but not too large that the pipe will not flow full or nearly full at Q_{25} to achieve orifice flow condition.

Try pipe dia, D = 1.8 m
Q_{25} = 5.0 m^3/s
Number of barrels = 1
HW / D = 0.925 (Design Chart 2.31)
Headwater, HW = 0.925 * 1.8 = 1.67 m
HW is slightly greater than channel flow depth 1.6 m at Q_{25}. O.K.

(c) Design the overflow side weir

A low hydraulic head for the side weir will require a long weir. A high hydraulic head will raise the downstream water surface level in the approach channel and cause more flow than Q_{25} to enter the pond. The design is balanced choice between the weir length and hydraulic head.

$$\text{Side weir flow, } Q_w = C_w * L_w\,[(h + v^2 / 2g)^{5/3} - (v^2 / 2g)^{5/3}] \qquad (4.2)$$

where:
Q_w = Weir capacity, m^3/s
C_w = Discharge coefficient (= 1.0)
L_w = Weir length, m
h = Hydraulic head on weir, m
v = Velocity of approach flow ($\approx$ 0 m^3/s)

Try h = 0.6 m
L_w = $7.0 / (1.0 * 0.6)^{5/3}$ = 16.4 m

Try h = 0.5 m
L_w = $7.0 / (1.0 * 0.5)^{5/3}$ = 22.0 m. Accept this.

Overflow velocity at weir, v_w, can be obtained by differentiating Equation 4.2:

$$v_w = (d(Q / L_w) / dh) = C_w * (5 / 3) * h^{2/3}$$
$$= 1.0 * (5 / 3) * 0.5^{2/3} = 1.05 \text{ m/s}$$

Place the weir with its crest at the elevation of the water surface of the approach channel at Q_{25}. Provide nominal riprap protection for the weir. For hydraulic design of the spillway below the side weir, see the section on Energy Dissipators in Chapter 5.

There is no recognized calculation method for determining the distance to separate the side weir from the pond inlet to minimize mutual effects of flow turbulence. A nominal distance equal to the length of the side weir appears to be reasonable.

Check that the hydraulic head of the side weir at Q_{reg} will not greatly increase the flow to the pond.

Inlet pipe dia, D	= 1.8 m		
HW at Q_{reg}	= 1.6 + 0.5	= 2.1 m	
HW / D	= 2.1 / 1.8	= 1.16	
Q / N	= 6.1 m^3/s		(Design Chart 2.31)
Flow to pond, Q	= 6.1 m^3/s	□Q_{25} O.K.	

Active Pool Elevation

The determination of the maximum elevations of the active pool, the permanent pool and the bottom of the pond should take the following factors into consideration:

- the pond outlet should not be submerged at Q_{25} by the receiving watercourse and, preferably, the pond should not be flooded at Q_{reg};
- the topography, groundwater table and soil conditions of the pond site are feasible for the proposed elevation and access to the pond by maintenance vehicles is also feasible;
- the proposed depths of the permanent and active pools can be achieved;
- the permanent pool should not be higher than the invert of the approach channel at the pond inlet, to prevent backflow of water from the pond to the channel;
- the active pool should not be higher than the mid-height of the installed inlet pipe to ensure that inlet control condition at the inlet pipe can be maintained; and
- any other special design requirements and site constraints.

This design example assumes that all these factors have been satisfied and that the maximum elevation of the active pool can be set at 0.5 m above the invert level of the installed inlet pipe. The corresponding elevation of the permanent pool will be at the invert level of the inlet pipe.

Sediment Forebay

To maximize the pond's settling capability, it is necessary to distribute the inflow as uniformly across the pond as possible. This is achieved by constructing a berm near the inlet end of the pond. See Figures (a) and (b). This flow distributor berm results in the formation of a *forebay* in the pond. This forebay can act as a first level settling zone for the coarser particles in the runoff.

The berm is located at a distance downstream of the pond inlet such that the jet resulting from the peak maximum inflow of 5.0 m^3/s will not shoot beyond the berm. This distance is calculated to be 80 m (approximately). The forebay so created contains approximately 20% of the total permanent pool volume. To enable the forebay to be emptied completely for de-silting, the berm may be constructed with an impermeable core.

The berm is submerged approximately 0.30 m below the permanent pool surface to create a more pleasing appearance of the pond surface. If practical, the upper portion of the berm (0.3 m) may be constructed to allow aquatic vegetation to take root. As such, the berm may help reduce safety concerns and make it even more aesthetically appealing.

Wet Pond Outlet

The outlet controls the release rate of the pond. Try a reverse-sloped pipe. See Figure (d). This pipe draws water from below the permanent pool level to minimize the potential for debris blockage. It can be designed as a small orifice that will empty the full active pool in the given detention time, when no inflow occurs during this period. Figure (e) shows the drawdown curve.

Detention time, T = 12 h
Active pool area, A = 18,000 m^2
Active pool depth, H = 0.5 m
Orifice coefficient, C_d = 0.55

Orifice area, a $= A * H^{0.5} / (0.5 * C_d * \sqrt{(2g)} * T * 3600)$ (4.3)
$= 18,000 * 0.5^{0.5} / (0.5 * 0.55 * \sqrt{(2 * 9.81)} * 12 * 3600$
$= 0.24\ m^2$

Outlet pipe dia, d $= \sqrt{(4 * 0.24 / \pi)}$ = 0.55 m.

Provide a 500 mm dia outlet pipe. If this size is not available, a 600 mm dia pipe may be considered and the detention time will be reduced to 10 h (Equation 4.3).

The 800-mm maintenance pipe allows draining of the permanent pool for maintenance. It is 1 m lower than the outlet pipe invert and directly connected to a maintenance hole. The maintenance pipe has a valve which is completely closed under normal operation.

Emergency Overflow Spillway

The pond is provided with an emergency spillway to prevent overtopping of the entire pond berm as an added safety measure for any unpredictable hydrologic event. The overflow weir is formed by a depression in the pond berm and designed for Q_{reg}. The weir crest is set at 0.5 m higher than the active pool elevation. Protect the weir with nominal riprap. For the design of the spillway below the weir, see the section on Energy Dissipators in Chapter 8.

Try hydraulic head over weir, H = 0.4 m
Broad-crested rectangular weir coef., C_d = 1.705
Required weir length, $L = Q / (1.705 * H^{3/2}) = 7.0 / (1.705 * 0.4^{3/2}) = 16.2$ m (8.74)

Figure (d): Wet Pond Outlet Arrangments

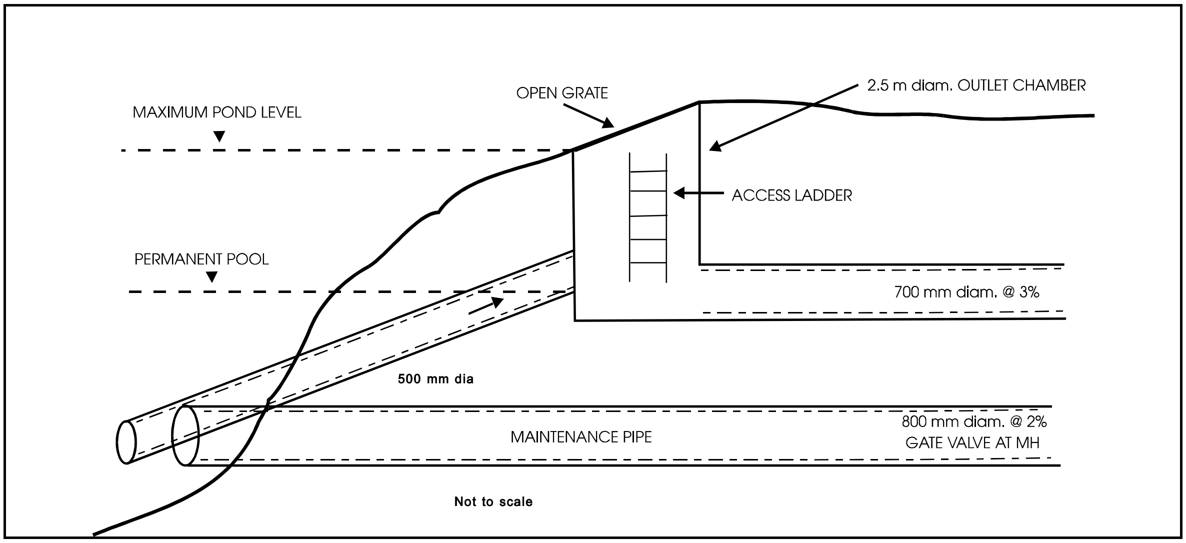
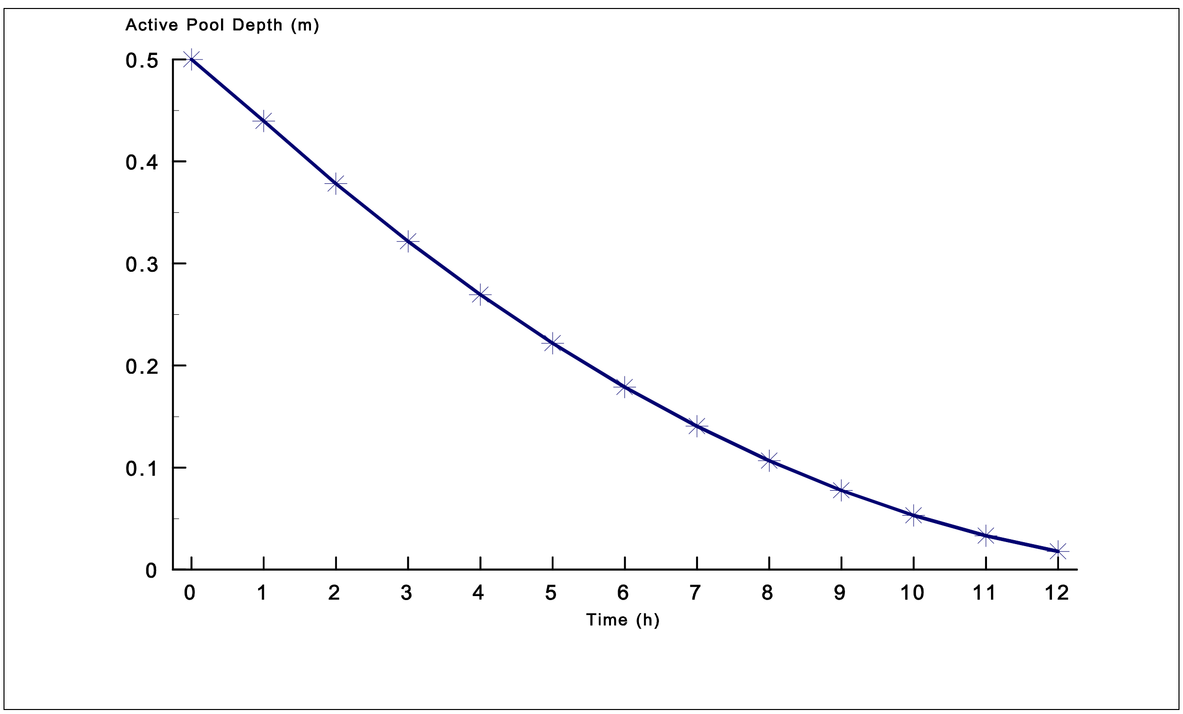


Figure (e): Outlet Drawdown Curve



Dry Ponds*

Dry Pond Features at a Glance

The main purpose of a dry pond is to manage **flow quantity**. It is used to control peak flow during floods or to accommodate limited capacity of drains that are required to convey flow in excess of their capacity. The way the pond achieves this objective is by detaining the flow and releasing it at a specific rate. The process of selection and development of a dry pond is normally dealt with in the preliminary design stage.

A dry pond does not have a permanent pool of water. It is designed to store the flow from the drainage area during a heavy storm. During a storm event, after the pond fills up, the pond releases the stored water continuously to a receiving drain or stream. The pond empties a short period (say 24 hours or less) after the storm. For a discussion of stormwater management, see Chapter 3.

In hydraulic design, a dry pond is different from a wet pond in that its capacity can be designed for a single storm (or several design storms). In contrast to wet ponds which are designed for frequent, light storms, dry ponds are designed for infrequent, heavy storms.

Design Example 4.12 illustrates the design procedure of a dry pond.

* This section is a continuation of Step 5 of the Detail Design Process.

Design Example 4.12
Hydraulic Design of Dry Pond

Required

Detail hydraulic design of a dry pond for stormwater quantity management for a proposed development site (called Phase III in Figure (a)).

Given

The following information is obtained from a preliminary design report of the project.

- The proposed development site is in Phase III, as shown in Figure (a). Phases I and II are already developed.
- The pre-development flow from Phase III and its upstream watersheds partly drains to an existing 525 mm dia. (original dimension 21 in.) railway culvert and partly to Phase II. See Figure (b). The existing Road 1 culvert drains to the railway culvert.
- The existing soil type of the site and upstream watersheds is mainly sandy loam, and the land use is corn cultivation. The topography is rolling.
- The proposed drainage management scheme is shown in Figure (c). It will use a dry pond for stormwater management. The pond site is at Area No. 3. Note that the post-development drainage areas are not all identical to those of pre-development and Area No. 1 is earmarked for development at some point in the future. Provision is needed in Phase III to accept this flow.
- A typical longitudinal section of the proposed pond and the existing railway culvert is shown in Figure (d). Note the control elevations.
- Design criteria for post-development conditions are as follows.
    - Flow into the railway culvert in 2 to 100-yr storms must not exceed the culvert capacity or the pre-development flows, whichever is the lesser, and must not overtop the railway track.
    - Use rainfall data of the Royal Botanical Gardens at Hamilton. For rainfall distribution, use 4-hr Chicago (Keifer and Chu method) hydrograph.
    - The railway culvert can be assumed to discharge under the free flow condition at all time (The culvert outlet is 0.5 m above the Regulatory flood elevation).
    - The proposed pond is allowed to bypass the Regulatory flow (Hurricane Hazel) from its emergency bypass.
- Tentative pond design: Surface area = 1.0 ha; side slope = 3:1; outlet = 0.5 m dia.

Figure (a): Site Location

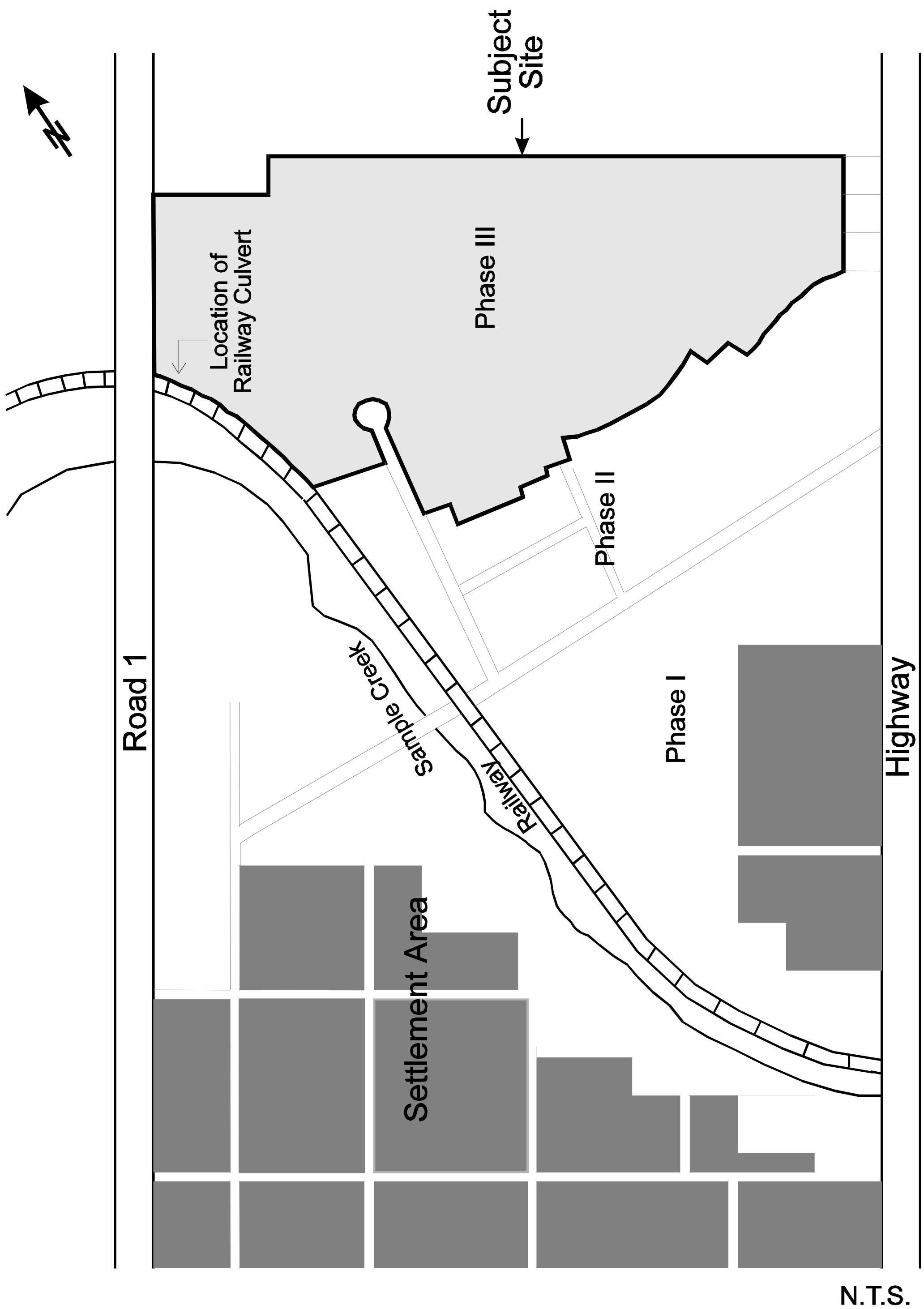


Figure (b): Pre-developed Drainage Areas

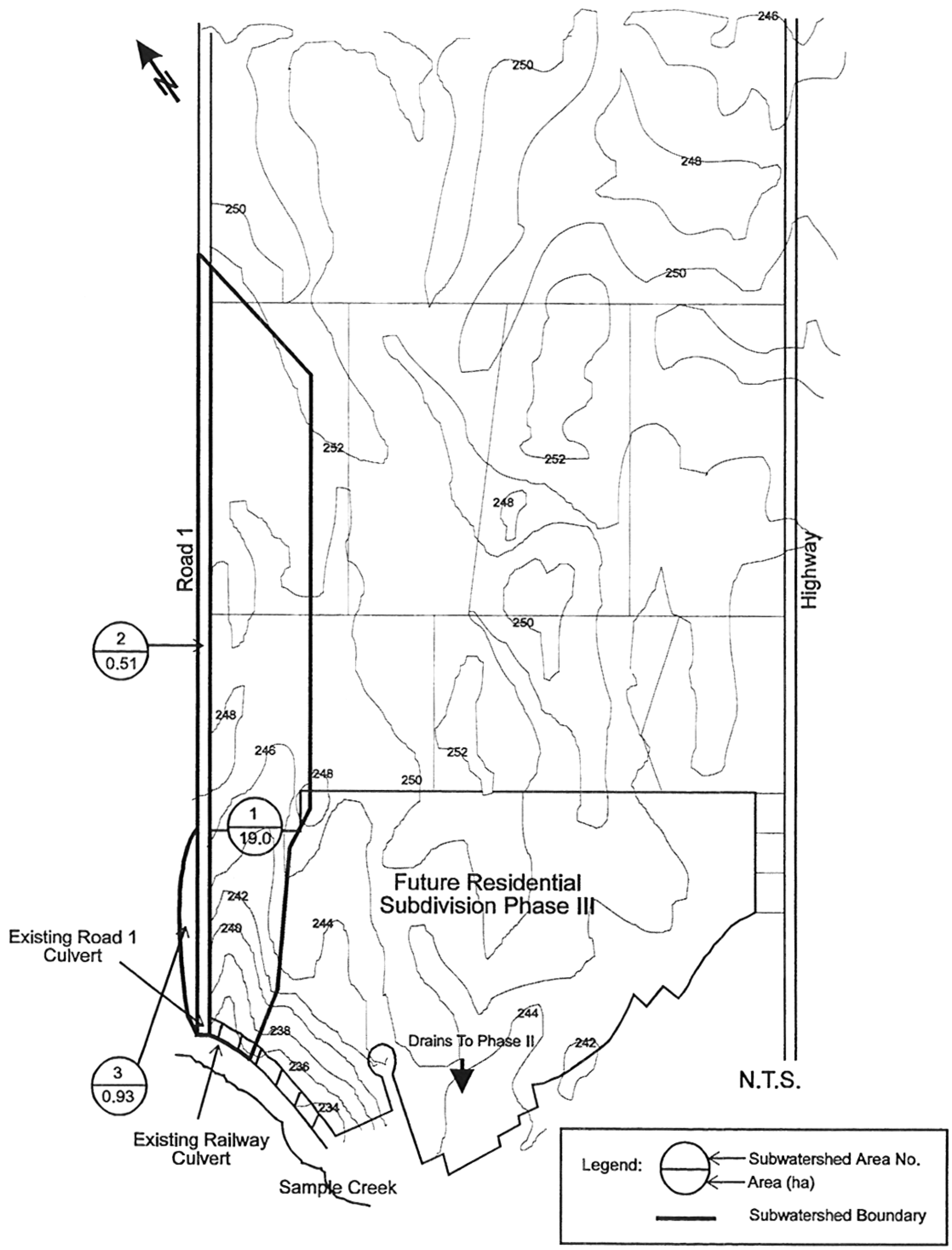


Figure (c): Post-developed Drainage Areas

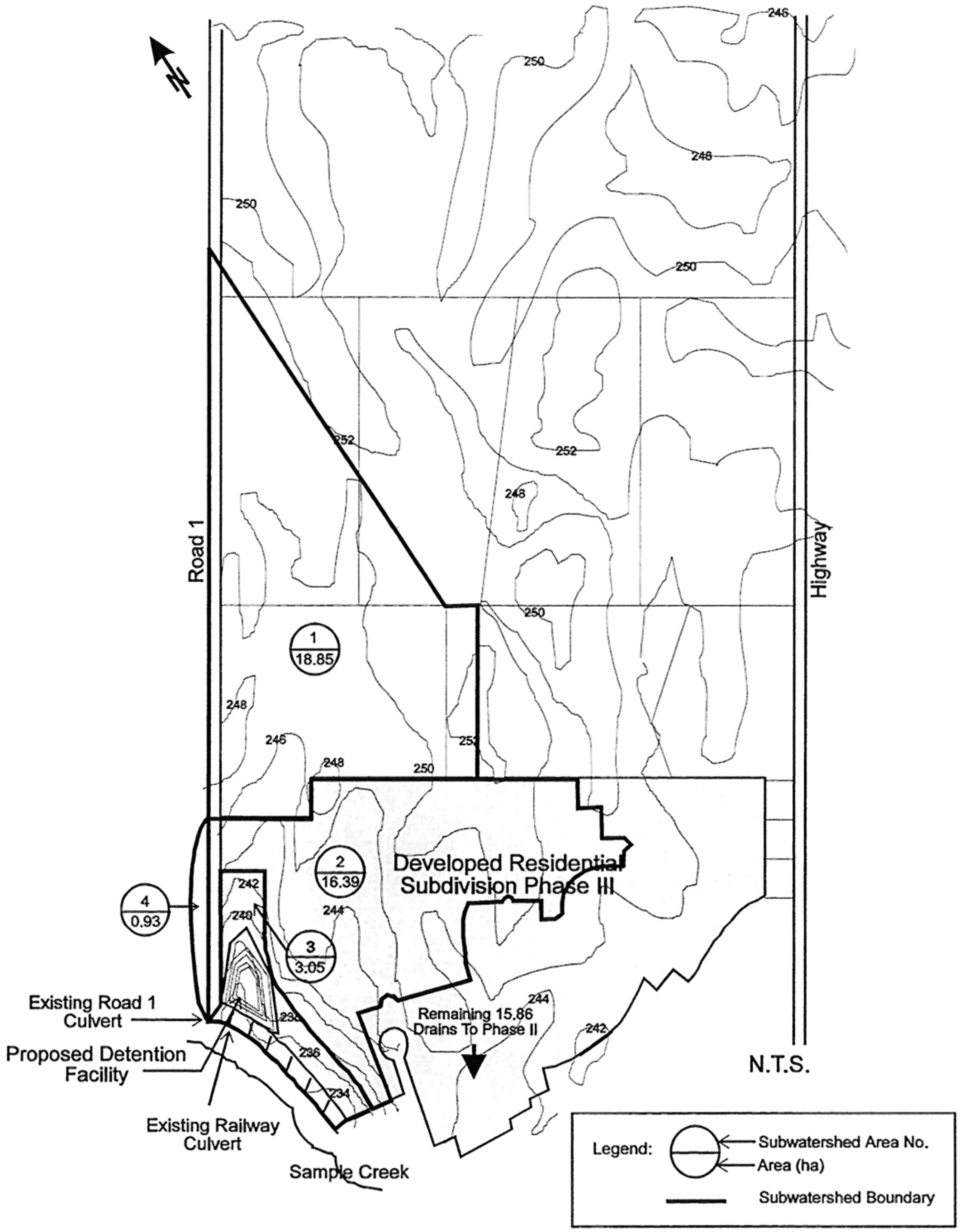
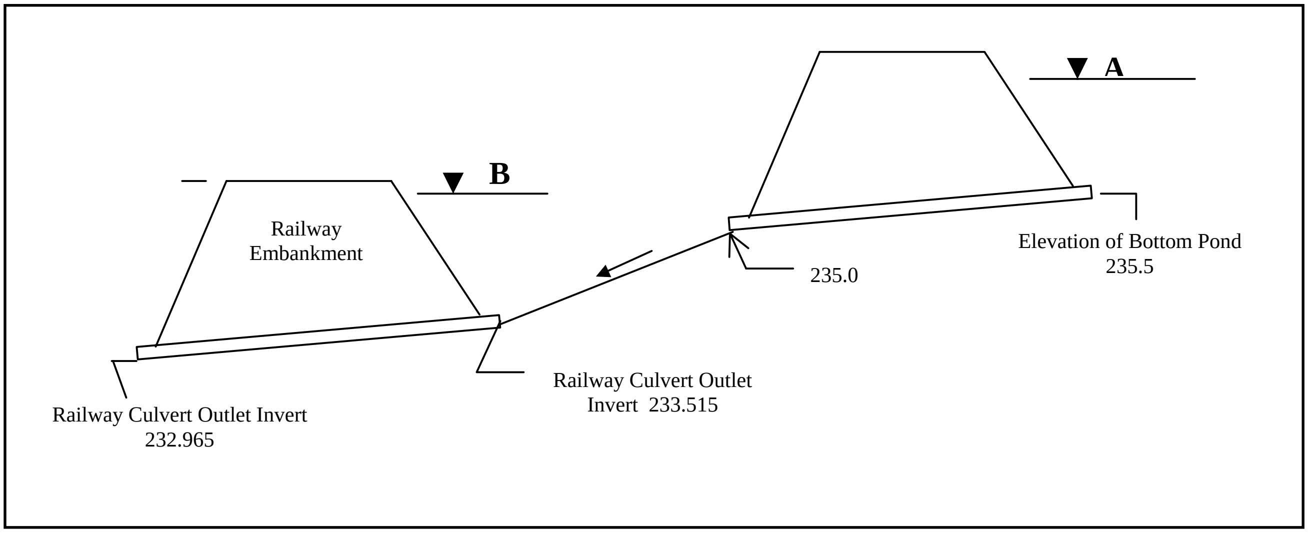


Figure (d): Typical Longitudinal Section of Pond



Preliminary Discussion

- X The required capacities of the pond and its outlet are interrelated. Increasing the outlet capacity will decrease the required pond capacity, and vice versa. However, the maximum design outlet capacity is controlled by the permissible discharge.
- X The appropriate capacities of the pond and outlet have to be determined by trial and error. The preliminary design results and the designer's experience will help reduce the number of iterations needed.
- X The calculation procedure shown in the Method section below represents one iteration.

Method

The main calculation tool used is OTTHYMO (also called INTERHYMO). It is used to determine the flows under three sets of conditions: (1) pre-development, (2) post-development (no pond) and (3) post-development (with pond). In each case, the range of design storms investigated are 2 to 100-yr and Hurricane Hazel. For working details of OTTHYMO, see the model's user manual.

In order to use OTTHYMO to investigate case (3), a discharge-storage relationship of the proposed pond must be developed and input to OTTHYMO. A spreadsheet is a convenient tool for developing the discharge-storage relationship. This design example will show how to create a spreadsheet using LOTUS 5.0.

1. Determine the existing capacity of the railway culvert.

The culvert capacity is limited by its inlet acting as an orifice (inlet control). Outlet control does not occur because the outlet is above the receiving water level at all time.

Railway embankment elevation = 235.778
Freeboard allowance = 0.3 m
Railway culvert invert elevation at inlet = 233.515
Culvert dia, d_r = 0.525 m
Max available head, h = 235.778 - 233.515 - 0.3 - (0.525 / 2) = 1.7 m
Orifice discharge coeff, C_d = 0.55
Capacity, $Q = 0.55 * (3.1416 * 0.525^2 / 4) * (2 * 9.81 * 1.7)^{1/2} = 0.68\ m^3/s$ (8.79)

Note: The flow can be obtained from a design chart. However, since the spreadsheet needs to use this formula for calculations in Step 5, it is advisable to use the same formula in Step 1 also for consistency.

2. Determine pre-development flow for each design storm.

(a) Adjust the watershed areas according to site conditions and additional data collected if applicable. Assume that Figure (b) has incorporated the refinement. Determine the hydrologic parameters required by OTTHYMO. The results are shown below.

Watershed ID No.	Area (ha)	Slope (m/m)	Length (m)	Time to Peak,T_p (h)	CN Curve No.
1	19.0	0.022	1300	0.27	77
2	0.51	0.013	1275	NA	99
3	0.93	0.023	460	0.10	81

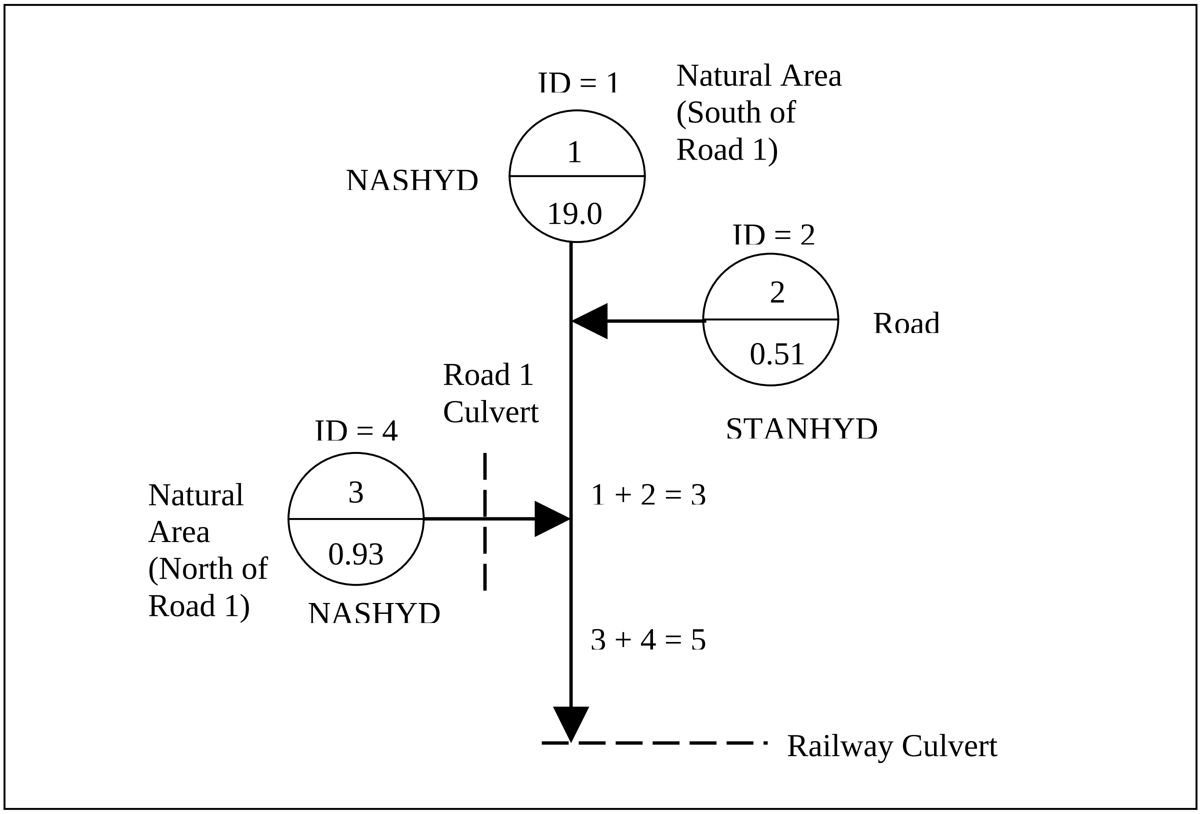
(b) Prepare flow schematic to be simulated by OTTYMO. See figure (e).

(c) Prepare OTTHYMO input file. Table (a) shows an example.

(d) Run OTTHYMO for each design storm. The flows, as determined at the inlet of the railway culvert, are shown in table (b). Note that each of the flows includes the flow from watershed are No.2 which is Road 1. This flow is small, about 0.06 m^3/s.

3. Repeat Step 2 to determine post-development flows with no pond control. Use Figures (c) and (f) for this step. The results are shown in Table (b).

Figure (e): OTTHYMO Pre-developed Drainage Schematic



4. Develop geometric details of the proposed pond.

   (a) Prepare a layout of the proposed pond on a contoured site plan of a suitable scale. Consider site conditions (e.g. topography, groundwater table, soil data and so on.) Consider detail design requirements (eg. Maintenance access, safety against accidental drowning and safety for small children, landscaping and so on.) Work with designers of other disciplines as necessary.

   (b) Calculate storage capacities of the pond at various water surface levels, from the layout and contours.

   (c) Try a pond outlet with pipe of a suitable size (0.45 m.) and inlet and outlet invert elevations. Figure (d) is used in this design example.

   Note: The size of the proposed pipe is small, because the results of Steps 1 and 3 clearly show that the railway culvert's capacity, 0.68 m^3/s, will be the limiting factor. Two questions may be asked: How do the pre-development flows greater than 0.68

m^3/s get to the creek if they cannot go through the railway culvert? Is the use of the railway culvert capacity as the limiting design criterion appropriate? It is beyond the scope of this design example to answer them.

Table (a): Sample OTTHYMO Input File

```
2
********************************
* Project: Phase III*
* File:    PRE10.dat        *
* Pre-Developed Conditions    *
********************************
START              TIME=0.0 hr
**********************************************
* Chicago Type Distribution, 10 year Storm*
**********************************************
CHICAGO STORM      IUNITS=2 TD=4.0 hr R=0.38 STD=5 min ICASE=1
                A=1065 B=5 C=0.788
***********************************************
* Runoff from South of Road 1 (Rural)*
***********************************************
DESIGN NASHYD      ID=1 NHYD=100 DT=10 min AREA=19.0 ha
                DWF=0 CN=77 TP=0.27 hr
                END=-1
**************************************
* Runoff from Road 1 (Urban)*
**************************************
CALIB STANDHYD      ID=2 NHYD=101 DT=10 min AREA=0.51 ha XIMP=0.99 TIMP=0.99
                DWF=0 LOSS=1
                FO=76.2 mm/hr FC=13.2 mm/hr DCAY=4.14 /hr F=0
                DPSP=1.5 mm SLPP=1.0% LGP=40.0 m MNP=.25 SCP=0
                DPSI=1.5 mm SLPI=1.3% LGI=1300 m MNI=.013 SCI=0
                END=-1
ADD HYD           ID=3 NHYD=102 IDONE=1 + IDTWO=2
***********************************************
* Runoff from North of Road 1 (Rural)*
***********************************************
DESIGN NASHYD      ID=4 NHYD=103 DT=10 min AREA=0.93 ha
                DWF=0 CN=81 TP=0.10 hr
                END=-1
ADD HYD           ID=5 NHYD=104 IDONE=3 + IDTWO=4
PRINT HYD          ID=5
FINISH
6
```

5. Calculate pairs of discharge-storage values for the pond of Step 4, using a spreadsheet. See Table (c).

Table (b): OTTHYMO Calculated Flows at Railway Culvert Inlet

Storm	Pre-development Flow (m3/s)	Post-dev Flow (No Pond) (m3/s)	Post-dev Flow (With Pond) (m3/s)
2-yr	0.53	2.59	0.35*
5-yr	0.97	4.21	0.46
10-yr	1.27	5.27	0.51
25-yr	1.71	6.89	0.60
100-yr	2.55	9.40	0.65
			* See note under 4(c)

Figure (f): OTTHYMO Post-developed Drainage Schematic

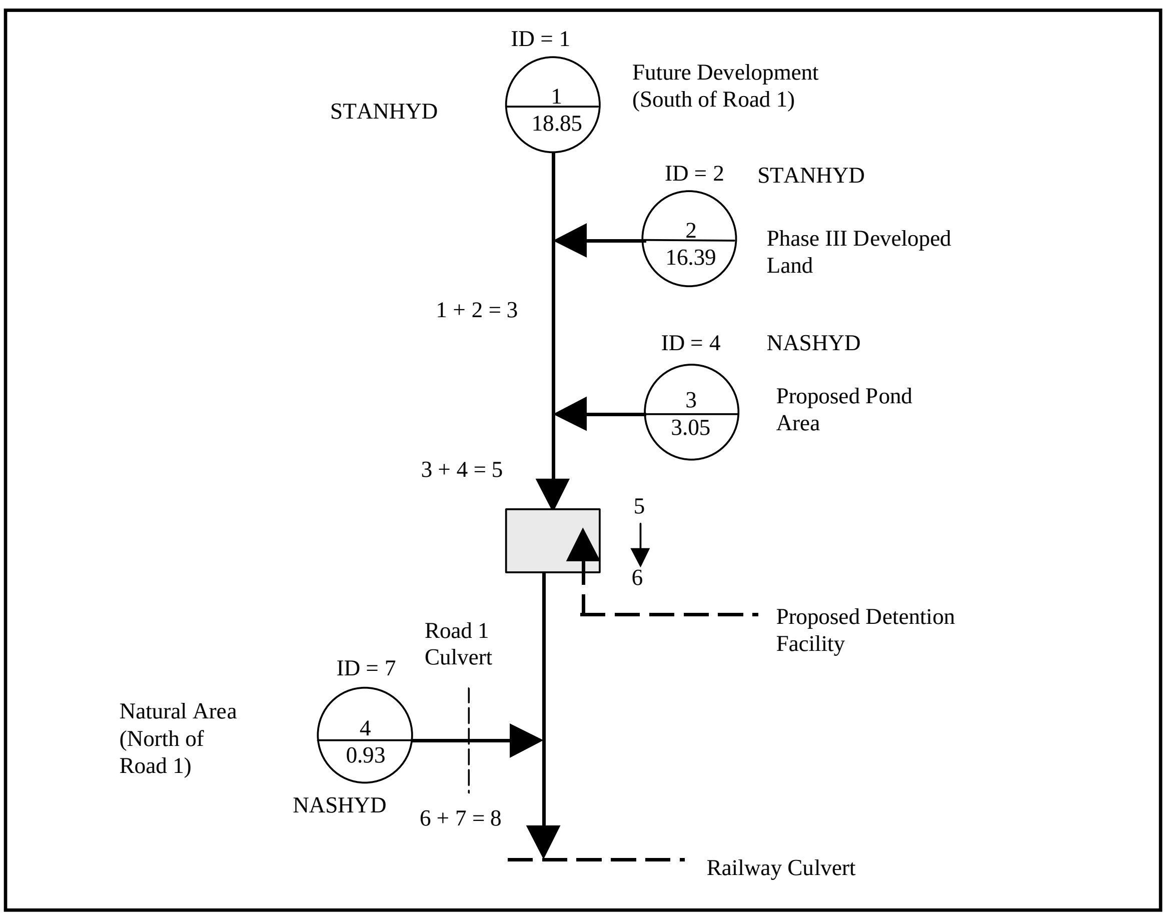


Table (c): Pond Discharge-Storage Relationship - Design Example 4.12

Pond Storage-Stage								**Row**
	Col 0	**Column 1 Elevation (m)**	**Column 2 Area (ha)**	**Column 3 Storage (ha.m)**				**1**
								2
Pond bottom elevation (m)	235.5	235.5	0.315	0.000				**3**
		236.0	0.350	0.153				**4**
		236.5	0.397	0.329				**5**
from site plan contours.		237.0	0.440	0.525				**6**
		237.5	0.486	0.746				**7**
	235.5	238.0	0.534	0.993				**8**
		238.5	0.610	1.305				**9**
		239.0	0.637	1.570				**10**
		239.5	0.800	2.120				**11**
Railway Culvert								**12**
								13
		Column 4	**Column 5**	**Column 6**				**14**
		Elev. B (m)	**CulvFlow (m3)**	**PondDisch(m3)**				**15**
Rail embankment elevation (m)	235.78	233.515	0.000	0.000	CulvInlet			**16**
Freeboard (m)	0.3	233.778	0.382	0.362	Top of rail culv			**17**
Upstream invert elevation (m)	233.52	234.015	0.460	0.440				**18**
Culvert dia, Dr (m)	0.525	234.265	0.531	0.511				**19**
Discharge coeff, Cd	0.55	234.515	0.592	0.572				**20**
		234.765	0.648	0.628				**21**
		235.015	0.700	0.680				**22**
		235.265	0.748	0.728				**23**
		235.478	0.787	0.767	Freebd Limit			**24**
Pond Outflow-Storage								**25**
								26
		Column 7	**Column 8**	**Column 9**	**Column 10**	**Column 11**	**Column 12**	**27**
		Outflow (m3)	**InletHead(M3)**	**OutletHead(m3)**	**ContHead(m3)**	**Elev. A (m)**	**Storage (m3)**	**28**
Upstream invert elevation (m)	235.5	0.000	0.000	0.000	0.000	235.500	0.000	**29**
Downstream elevation (m)	235	0.362	0.873	0.378	0.873	236.148	0.205	**30**
Pipe dia, Dp (m)	0.45	0.440	1.292	0.459	1.292	236.567	0.355	**31**
Pipe length, Lp (m)	29	0.511	1.736	0.533	1.736	237.011	0.530	**32**
Pipe slope, Sp	0.0172	0.572	2.183	0.597	2.183	237.458	0.727	**33**
Manning's n	0.013	0.628	2.631	0.656	2.631	237.906	0.947	**34**
Discharge coeff, Cd	0.55	0.680	3.080	0.710	3.080	238.355	1.215	**35**
Entrance head loss coeff, Ke	0.5	0.728	3.531	0.760	3.531	238.806	1.467	**36**
		0.767	3.915	0.800	3.915	239.190	1.779	**37**

The following explains how to create Table (c). The spreadsheet may start at any cell. In the following discussion, the column and row numbers in the table are used as reference points.

Col. 1-3 Enter the pond storage-stage values obtained from Step 4(b).

Col. 4 Enter 233.515 (culvert inlet invert elevation) in Row 16; 233.778 (top of railway culvert) in Row 17; and 235.478 (maximum freeboard elevation) in Row 24. For each of the remaining rows, enter a convenient incremental value. This table uses increments of 0.25 added to the value 233.515.

Col. 5 Enter 0 in Row 16. Enter Eq. 8.79 (the same as in Step 1) in Row 17, where the head, h = Col4Row17 - Col4Row16, culvert dia is $Col0$Row19, and coefficient of discharge is $Col0$Row20. Copy the formula in Col5Row17 to the remaining rows of this column.

Col. 6 Enter 0 in Row 16. Enter (Col5Row17 - 0.06) in Row 17. The value 0.06 is the approximate flow from Road 1. Copy the formula in Col6Row17 to the remaining rows of this column.

Col. 7 Copy Col. 6 to Col. 7 for all the rows in the column.

Col. 8 This column represents the hydraulic head required to produce the pond discharge in Col. 7 if the pond outlet is under inlet control. The equation is the same as in Step 1 but re-arranged:

h = (Q / (Cd * 3.1416 * Dp2 / 4)) / (2 * 9.81)

Enter 0 in Row 29. Enter the re-arranged formula in Row30, where Cd and Dp are obtained from Col. 0 (use absolute addresses) and Q is Col7Row30. Copy the formula in Col8Row30 to the remaining rows of this column.

Col. 9 This column represents the hydraulic head required to produce the pond discharge if the pond outlet is under outlet control. The head is,

$$H = \left(1 + Ke + \frac{19.6 * Lp * n2}{(Dp / 4)\ 1.333} \right) \frac{*V2}{2g}$$

where Ke, Lp, Dp and n are obtained from Col. 0 (use absolute addresses) and v = Q/(3.1416 * Dp2/4). Enter the equation into Row 30, where Q is Col7Row30. Copy Col9Row30 to the remaining rows of this column.

Col 10 The value for Row 29 is the greater of Col8Row29 and Col9Row29. Use the @IF spreadsheet command to construct the content of Col10Row29. Copy this row to the remaining rows of this column.

Col 11 The value for Row 29 is: if [Col4Row16 > $Col0$Row30 (this is the invert of the pond pipe at outlet), Col11Row29 = Col4Row16 + Col10Row29 (that is: Elevation B + control head)], else Col11Row29 = $Col0$Row29 + Col10Row29 (that is: invert of the pond pipe at inlet + control head).

Col 12 The values are obtained by interpolation of a pair of elevations in Col. 1 with the values of the corresponding pair of storage in Col. 3. Take Col12Row30 for example. Its value is the storage corresponding to Elevation A = 236.148. Determine, manually, in Col. 1 that 236.148 lies between 236.00 and 236.50 in Rows 4 and 5 respectively. Their corresponding storage values are 0.153 and 0.329 in Col. 3. Therefore, in mathematical terms,

Col12Row30 = 0.153 + (0.329 - 0.153) * (236.148 - 236.0)/ (236.5 - 236.0)

In spreadsheet terminology,

Col12Row30 = Col3Row4 + (Col3Row5 - Col3Row4) * (Col11Row30 - Col1Row4)/(Col1Row5 - Col1Row4)

Repeat this step for each of the remaining rows of Col. 12. Do not simply copy Col12Row30 to the remaining rows.

Note: It is possible to make the spreadsheet do the interpolation automatically. But this requires the use of macros. It is not worth the effort to write macros for the spreadsheet to be used for one design. If the macros are to be applicable for any design, then the spreadsheet format must be standardized and must be able to accommodate a variety of pond configurations, inlet and outlet conditions, and so forth. Such a spreadsheet would be unwieldy.

6. Repeat Step 3, but this time is for post-development with pond control. The pond control can be done by including the ROUTE RESERVOIR command in the OTTHYMO input. The required outflow-storage values for input to OTTHYMO will be the pairs of values from Col. 7 and 12 of Table (c).

7. Review the results of Step 6 to see if the design criteria are satisfied and whether there is substantial room for reducing the pond size and/or increasing the pond outlet size. Repeat some or all of Steps 1 to 6 as desired.

8. Do a hydraulic design of the emergency bypass.

   (a) Repeat Step 3 but using Hurricane Hazel rainfall data to determine the magnitude of the Regulatory flow.
   (b) Follow the method given in Design Example 4.11 to determine the dimensions of the emergency bypass weir.

9. Design erosion control measures for the inlet, outlet and emergency bypass of the pond. The methods are provided in the Energy Dissipators section in Chapter 5.

10. Design inlet and outlet chambers of the pond. See Design Example 4.11.

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Appendix 4A: Summary of Design Methods and Formulas

Table 4A.1: Summary of Commonly Used Methods

Table 4A.2: Summary of Formulas

Figure 4A.1: Summary of Commonly Used Methods

Surface Drainage Component	Design Task	Hydrology Methods	Hydraulics Methods
(1) Roadside Ditches	Minor flow (usually 2 to 10-yr storm).	(a) IDF rainfall charts, plus manual calc of Rational method, or (b) MTO Rational Drainage model.	(a) Design charts, or manual calc of Manning's formula, or (c) MTO Open Channel model.
(2) Roadside Ditches	Major flow.	(a) IDF rainfall charts, plus manual calc of runoff (SCS or Keifer&Chu) and flow routing(Muskingum), or (a) OTTHYMO (b) model, or (c) MIDUSS model.	(a) MTO Open Channel model plus HEC-2 model plus shear stress formulas for erosion control.
(3) Roadside Ditches	Minor flow conveyance and sedimentation.	(a) Same as (1).	Same as (1) plus weir formula to determine storage and overflow.
(4) Single Sewer Pipe	Free outlet. No M.H. energy loss. Pipe not surcharged.	(a) IDF rainfall charts, plus manual calculation of Rational method or (b) MTO Rational Drainage model.	(a) Design charts or manual calculation of Manning formula, or (d) MTO Storm Sewer model.
(5) Storm Sewer Network	Free outlet. Pipes not surcharged.	Same as (4).	Same as (4) plus appropriate allowance for energy losses at M.H. (Cont'd)

Table 4A.1: Summary of Commonly Used Methods (Cont'd.)

Surface Drainage Component	Design Task	Hydrology Methods	Hydraulics Methods
(6) Storm Sewer Network	Tailwater effects or submerged flow (water elev in M.H. higher than crown of sewer pipe).	Same as (5).	(a) Design charts or manual calc with allowance for downstream water elev, and M.H. energy losses, or (b) MTO Storm Sewer model.
(7) Pavement Drainage	Various r.o.w. widths and grades. Curbed and uncurbed pavement.	(a) IDF rainfall charts, plus manual calc of Rational method, or (b) MTO Rational Drainage model.	(a) Manual calc of Manning's formula, or (b) Design charts, or (c) MTO CBSpace model.
(8) Wet Ponds and Extended Dry Ponds*	Various storm conditions.	See right column.	(a) Probability distribution method (e.g. MTO SUDS Extension model) and associated rainfall statistics, or (b) Continuous simulation (e.g. SWMM plus MTO SWMM Extension models).
(9) Dry Ponds* * Best see Design Examples.	Single design storm.	Hydrologic simulation (e.g. OTTHYMO).	(a) Stage-discharge calculation.

Table 4A.2: Summary of Formulas

Rainfall Intensity Equation No.*

$$I = \frac{A}{(t + B)^c} \qquad (8.1)$$

where:
I = Average rainfall intensity, mm/h
t = Rainfall duration, min
A, B, c are coefficients

Chicago Storm Distribution (Keifer-Chu Method)

$$I_p = \frac{A}{()t + B)^c} \qquad (8.7)$$

where:
I_p = Peak rainfall intensity, mm/h
Δt = Time step used in rainfall distribution, min
A, B, c are constants

Composite Runoff Coefficient

$$C = \frac{(A_1 * C_1 + A_2 * C_2 + ... A_n * C_n)}{(A_1 + A_2 + ...A_n)} \qquad (8.10)$$

where:
C = Composite runoff coefficient
A_i = Area of catchment i, ha
C_i = Runoff coefficient of catchment i

Watershed Slope

$$S_w = \frac{100 * ()h - h_f)\ \%}{(0.75 * L - L_f)} \qquad (8.13)$$

where:
S_w = Subwatershed slope, %
Δh = Difference in elevation between 85% and 10% points of watershed slope, m
h_f = Sum of heights of rapids and waterfalls, m
L = Total length of watershed main channel, m

*(8.1) indicates the equation is reproduced from Equation 8.1 in Chapter 8.

Table 4A.2: Summary of Formulas (Cont'd)

Watershed Time of Concentration
(a) Bransby-Williams Equation

$$T_c = \frac{0.057 * L}{S_w^{0.2} * A^{0.1}} \quad (8.15)$$

(b) Airport Formula

$$T_c = \frac{3.26 * (1.1 - C) * L^{0.5}}{S_w^{0.33}} \quad (8.16)$$

where:
T_c = Time of concentration, min
L = Watershed length, m
S_w = Watershed slope, %
A = Watershed area, ha

Rational Method

$$Q = 0.0028 * C * I * A \quad (8.19)$$

where:
Q = Peak runoff rate, m^3/s
C = Composite runoff coefficient
I = Rainfall intensity, mm/h
A = Drainage area, ha

Time to Peak (Hymo Method)

$$T_p = 0.0086 * A^{0.422} * S^{-0.46} * (L / W)^{0.133} \quad (8.34)$$

where:
T_p = Time to peak, h
A = Drainage area, ha
S = Watershed slope, m/m
L = Watershed channel length, m
W = Watershed width, m

Table 4A.2: Summary of Formulas (Cont'd)

Continuity Equation

$$Q_1 = A_1 * V_1 = Q_2 = A_2 * V_2 = ... = Q_n = A_n * V_n \quad (8.48)$$

where:
Q_i = Flow rate at cross section i, m^3/s
A_i = Area of cross section i, m^2
V_i = Flow velocity at cross section i, m/s

Energy Equation

$$h = (y_1 + \frac{V_1^2}{2 * g}) - (y_2 + \frac{V_2^2}{2 * g}) + (Z_1 - Z_2) \quad (8.50)$$

where:
h = Head loss, m
y_i = Flow depth at cross section i, m
V_i = Flow velocity at cross section i, m/s
g = Gravitational acceleration, m/s^2
Z_i = Invert elevation at cross section i, m

Froude Number

$$F_r = \frac{Q}{A * \sqrt{(g * y_m)}} \quad (8.55)$$

where:
F_r = Froude number
Q = Flow rate, m^3/s
A = Flow cross sectional area, m^2
g = Gravitational acceleration, m/s^2
y_m = Hydraulic mean depth, m (For open channel, y_m = Flow area / top width of flow)

Manning Equation

$$V = \frac{1}{n} * R^{2/3} * S^{1/2} \quad (8.66)$$

where:
V = Flow velocity, m/s
n = Manning's roughness coefficient
R = Hydraulic radius, m
= Flow area / wetted perimeter

Table 4A.2: Summary of Formulas (Cont'd)

Composite Manning's Roughness Coefficient

$$n_c = \frac{\sum (A_i * R_i^{2/3} * n_i)}{\sum (A_i * R_i^{2/3})} \quad (8.68)$$

where:
n_c = Composite Manning's roughness coefficient
A_i = Area of cross section i of channel, m^2
R_i = Hydraulic radius of cross section i of channel, m
n_i = Manning's roughness coefficient of cross section i

Rectangular Weir

$$Q = C_d * L * H^{3/2} \quad (8.74)$$

where:
Q = Flow over weir, m^3/s
L = Weir length, m
H = Hydrostatic head above weir, m
C_d = Weir coefficient

Orifice (Small Diameter)

$$Q = C_d * A * \sqrt{(2 * g * H)} \quad (8.79)$$

where:
Q = Flow rate at orifice, m^3/s
C_d = Orifice coefficient
A = Cross sectional area of flow, m^2
H = Hydrostatic head above orifice, m

Head Loss through Pipe Flow

$$H = \left[1 + K_e + \frac{19.6 * n^2 * L}{R^{4/3}}\right] * \frac{V^2}{2g} \quad (8.84)$$

where:
H = Head loss, m
K_e = Entrance loss coefficient
n = Manning's roughness coefficient
L = Length of pipe, m
R = Hydraulic radius of pipe, m
V = Flow velocity, m/s
g = Gravitational acceleration, m/s^2

Table 4A.2: Summary of Formulas (Cont'd)

Critical Shear Stress of Non-cohesive Lining Material

$$\vartheta_c = 0.0642 * D_{50} \qquad (5.31)$$

where:

ϑ_c = Critical shear stress of the lining material, kg/m^2
D_{50} = Median particle size of lining material, mm

Modified Manning Equation for Gutter Flow

$$Q = \frac{0.375 * S_O^{0.5} * d^{2.667}}{n * S_x} \qquad (4.1)$$

where:

Q = Gutter flow rate, m^3/s
S_0 = Gutter slope, m/m
d = Gutter flow depth at curb, m
n = Manning's roughness coefficient
S_x = Pavement cross fall, m/m

Side Weir (U.S. EPA, 1989)

$$Q_w = C_w * L_w * [(h * V^2 / 2g)^a - (V^2 / 2g)^a] \qquad (4.2)$$

where:

Q_w = Discharge over weir, m^3/s
C_w = Discharge coefficient = 1.0
L_w = Weir length, m
h = Driving head of weir flow, m
V = Approach velocity . 1.0, m/s
a = Weir exponent = 5/3 for side weir

Pond Outlet Pipe Size

$$a = \frac{A * H^{0.5}}{0.5 * C_d * \sqrt{(2g)} * 3600 * T} \qquad (4.3)$$

where:

a = Cross sectional area of outlet pipe, m^2
A = Surface area of active pool, m^2
(Assume the pool's surface area is constant for all depths of pool)
H = Total depth of active pool, m
C_d = Orifice coefficient of outlet pipe
T = Design detention time of active pool, i.e. time required to empty the full active pool, assuming no inflow during this period, h

Appendix 4B: Design Forms

Table 4B.1: Sewer Design Form

Table 4B.2: Inlet Spacing Design Form

Table 4B.1: Sewer Design Form

STORM SEWER DESIGN CALCULATIONS

HWY. NO. ______________

W.P. NO. ______________

DESIGNED BY ______________ DATE ______________

CHECKED BY ______________ DATE ______________

DESIGN FREQUENCY ______________

RAINFALL STATION(S) ______________

DESIGNED 'n' ______________

MINIMUM COVER ______________

LOCATION				DRAINAGE AREA				RUNOFF			PIPE SELECTION						Minor Losses m	PROFILE				REMARKS
FROM		TO		A	C	A*C	Cumul. A*C	Cumul. Tc	i	Q	Pipe L	Pipe So	Pipe D	Actual Capacity (full)	V (full)	Time of Flow		Fall in Sewer	UPSTREAM		D/S	
MH or CBMH No.	Sta.	MH or CBMH No.	Sta.	ha				(Ti - ___) min	mm/h	m^3/s	m	m/m	m	m^3/s	m/s	min		m	Surface Elev. m	InvEL m	InvEL m	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23

Table 4B.2 Inlet Spacing Design Form

Sewer Inlet Spacing Calculation

Storm Frequency (Yr)
Rainfall Station
Rainfall Intensity (mm/h)
Allowable Spread (m)
Hwy No. ____________________
W.P. No. ____________________
Designed By ________ **Date** ________
Checked By ________ **Date** ________

Allow. Max. Spacing (m)
Curb & Gutter Type
Inlet Type
Inlet Time (min)

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
		Inpyt Data										**Calc.Data**	**Design Results**						
Line No:	Station	**Area Widths, W (m)**				**Runoff Coef., C**				Gutter Grade So (m/m)	**Cross-fall Sx (m/m)**	Com-posite C	Gutter Flow Qg (m3/s)	Local Runoff Qr (m3/s)	Inlet Spacing L (m)	Calc. Spread T (m)	Inlet Capacity Qi (m3/s)	Carry Over Qc (m3/s)	
		W1	W2	W3	W4	C1	C2	C3	C4										
1																			
2																			
3																			
4																			
5																			
6																			
7																			
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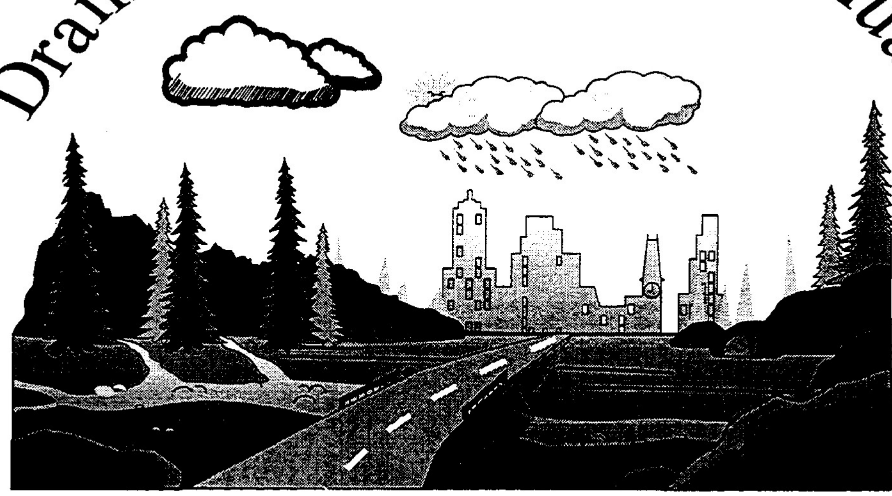
Ministry of Transportation

Ontario

MTO Drainage Management Manual



Chapter 5

Bridges, Culverts and Stream Channels

Drainage and Hydrology Section
Transportation Engineering Branch
Quality and Standards Division

5 Table of Contents

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Purpose of this Chapter

The purpose of this chapter is to provide a process and methods for detailed hydraulic analysis and design of bridges, culverts and stream channels. This is done with the aid of design examples.

The subjects covered in this chapter are organized as shown on Figure 5.1. As a starting point, it is best to begin with the determination of the waterway opening. Appendix 5B: Typical Bridges, Culverts and Transition Structures may be read for an introductory discussion on the features of these structures. More experienced designers may wish to use selective subjects of the chapter as desired.

Considerations that are more suitable to the preliminary design stages are presented in Chapter 3 of Part 1; theoretical background in Part 3; design charts in Part 4 and bridge deck drainage in Chapter 4.

Figure 5.1: Detailed Hydraulic Design

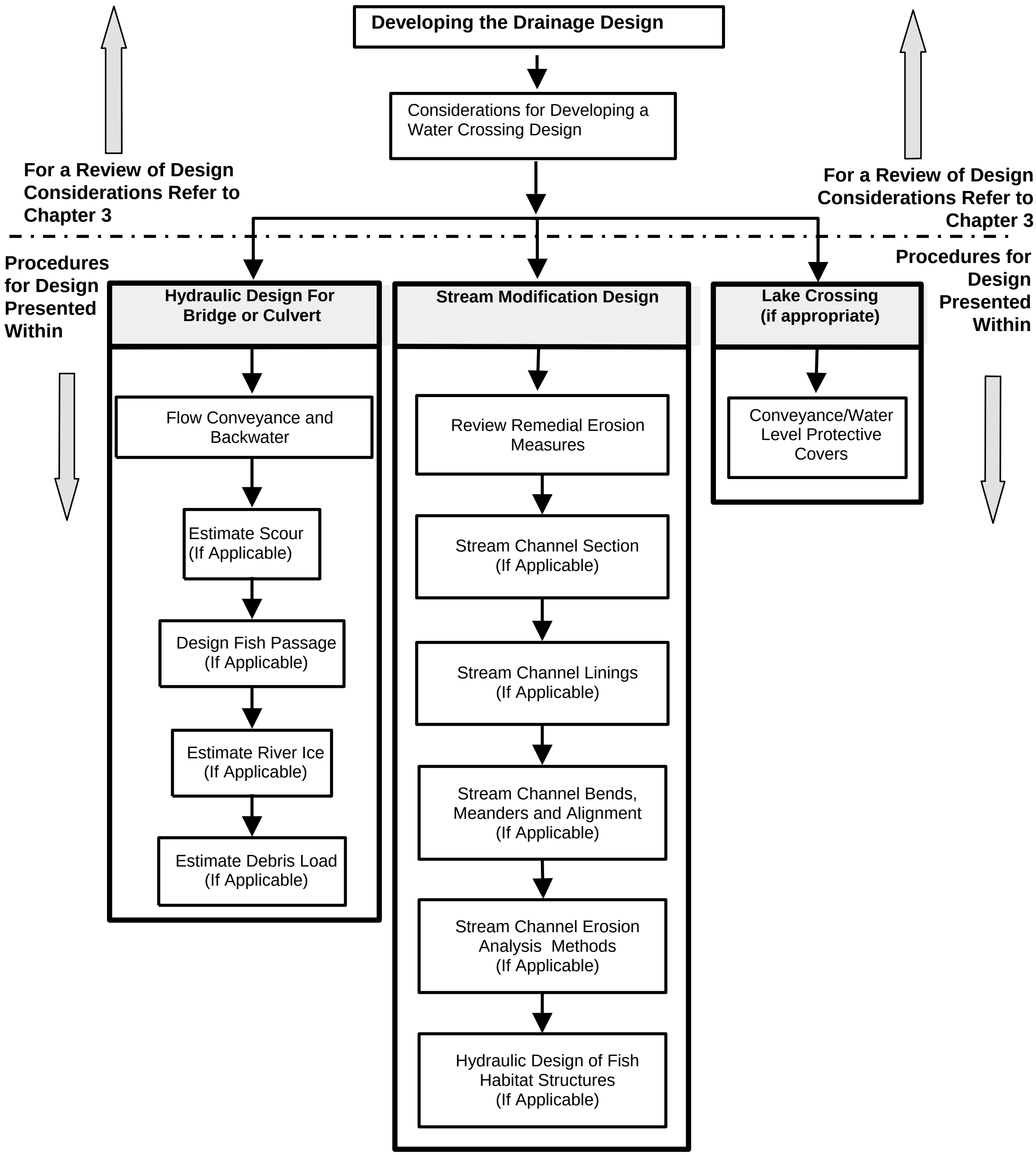
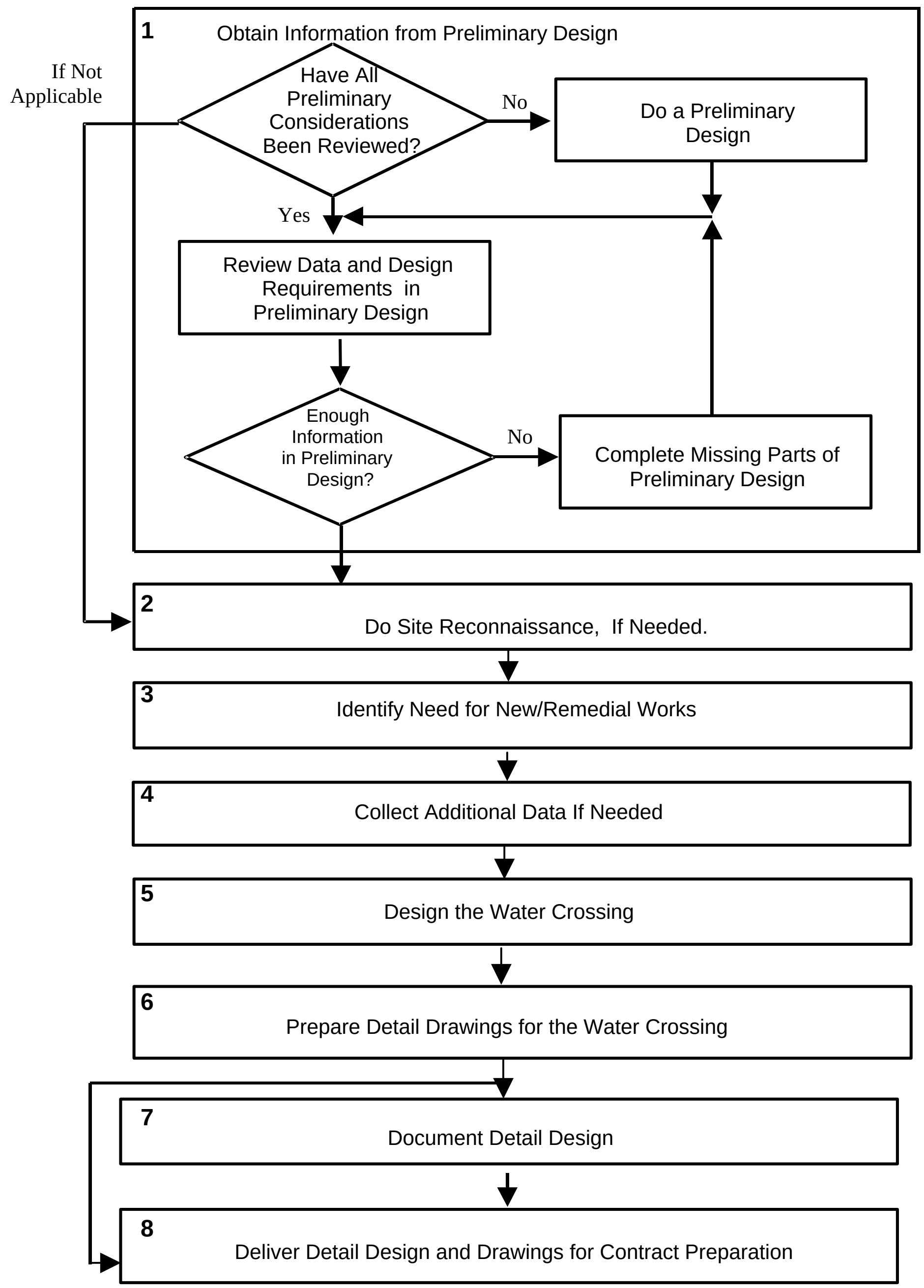


Figure 5.2: Enlargement of Detail Design Step of Figure 1.1



Detailed Hydraulic Design

The procedure presented in Figure 5.2 can be used to guide the detailed hydraulic design of bridges, culverts and stream channel works for transportation projects. It is recognized that the nature and scope of a particular highway project affects the actual sequence of steps of planning and design. However, for organizational purposes, the procedure presented in Figure 5.2 is used to outline the detailed hydraulic design of bridges, culverts and stream channels. Furthermore, the procedure presented in Figure 5.2 is generic and flexible and can be adjusted to suit the planning and design procedure of the highway project. Key steps are discussed below.

Step 1: Obtain Information from Preliminary Design (if applicable)

Step 1 is critical in that it helps a designer to see whether the project is ready for detail design. The reason for this check is that detail design may only be one stage of the entire highway planning and design process as illustrated in Figure 1.1 of Chapter 1. As seen in Figure 5.2, the preliminary design stage will include tasks that will consider and resolve all major design issues concerning the highway project, including drainage. These tasks should be accomplished before the detail design stage starts and can be completed even if the preliminary and detail design stages are done in one assignment. For guidance on the considerations and tasks appropriate for the preliminary stages of drainage design, refer to Chapter 3 in Part 1 of this manual.

For highway projects where preliminary design is appropriate and has been completed, a preliminary drainage design report is expected to indicate whether the major issues shown in Table 5.1 apply or not as all issues should be addressed. Where major issues apply, the report is expected to take the actions listed in Table 5.1.

It is not practical to make Table 5.1 an exhaustive list. Therefore, a designer must ascertain whether any unlisted issues may apply to the particular task.

If a designer determines that some major issues are not addressed, or not addressed satisfactorily, it is prudent not to proceed further with the design. Instead, completion of the preliminary design should be considered. Refer to Chapter 3 in Part 1 of this manual to review the considerations appropriate for the preliminary stages of design.

Table 5.1: Actions Expected from Preliminary Design Stage

Possible Major Issues Identified and Resolved	Design Criteria Established

• Hydrologic or hydraulic impacts.
• Scour problem.
• Fishery impacts.
• River ice/debris flow problem.
• Erosion problem.
• Stream stability complications.
• Flood plain protection problem.
• Riparian rights concerns.
 	
Water Crossings
 	
• Tentative location and alignment of crossing identified.
• Choice made between a bridge and culvert.
• Preliminary layout of abutments relative to stream cross section developed.
• Tentative overall opening width determined.
• If bridge: tentative number of spans determined.
• If culvert: tentative number of cells determined.
 	
Channel Diversion/Bank Protection
 	
• Preliminary diversion layout proposed, if applicable.
• Remedial erosion protection works proposed, if applicable.
• Tentative scope and nature of works defined.
• Tentative layout of fish habitat structures, if applicable.
	• Fish passage, if applicable:
	  • Migration period.
	  • Allowable delay in migration.
	  • Max. fish swimming performance.
	  • Special provisions, if any, for resting and substrate material.
	• Fish habitat structures, if applicable:
	  • Special hydrologic/hydraulic provisions, if applicable.
	• Design flow frequencies (there may be more than one required.)
	• Allowable maximum backwater elevation.
	• Whether or not pressure flow through a crossing is allowed.
	• Whether or not relief flow at or near a crossing is allowed.
	If relief flow is allowed:
	  • Max. allowable backwater elevation.
	  • Design flow frequency.
	  • Location of relief flow path.
	  • Special provisions, if any, to protect upstream/downstream land, the roadway, highway safety, public flood safety and use of highway for emergency services during a relief flow event.
	• Special provisions associated with stream diversion, if applicable.
	• Special Provisions, if any, for bank protection.

Normally, a certain amount of data will have been collected and used in addressing the major issues in a preliminary design. Table 5.2 is an example of the typical data used. If some data is not available or appears to be incomplete or questionable in quality, the designer should ascertain whether to seek clarification before proceeding further with the design.

Table 5.2: Typical Data Used in Preliminary Design

The Reach of Stream Containing the Project	Data of Existing Water Crossings
• Topographic maps and mosaics. • Interpretation of stream geomorphology and land surface features and texture. • Flood plain mapping. • Stream flow records and flow frequency analysis results. Alternatively, watershed maps, precipitation records and hydrologic modelling results. • Fish data related to hydrologic/hydraulic design	• Upstream and downstream crossings. • Crossing types and arrangements of abutments and piers. • Locations of crossings relative to stream cross sections. • Dimensions of openings of crossings. • Control elevations of crossings, e.g. crown of culvert, soffit of low chord of bridge. • Allowable and historical maximum backwater elevations. • Any past major problems with crossings.

Step 2: Do Site Reconnaissance

Designers are, themselves, the best judges to determine whether a site reconnaissance is needed. Generally, it is desirable to have a site reconnaissance, especially if any of the following circumstances exist:

- The designer is not familiar with site conditions.
- Complex topography exists and/or a complex road profile is involved.
- Some reprovisioning works may be needed, for example stream bank restoration necessitated by erosion.
- The proposed works are hydraulically complex.
- There are possible concerns of neighbouring property owners about the project.
- Potential or past concerns of the public arising from flooding, stream geomorphology changes, river ice or debris flow problems, or serious abutment scour.
- Follow up on the recommendations or advice of the preliminary report.

Important observations made in a site reconnaissance should be documented for future actions and reference.

Step 3: Identify Need for New/Remedial Works

Step 3 is a check. Changes in the scope of work may be required due to any information obtained in Step 2: Site Reconnaissance.

Step 4: Collect Additional Data

Table 5.3 is a summary of possible additional data requirements for detail design. For more details, see Appendix 5A.

Table 5.3: Possible Additional Data Requirements

• Site plan, profiles and cross sections of highway project. • Surveys of site of proposed works, including stream profiles, invert profile, selected stream cross sections for sufficient distance upstream and downstream of the site.	• Site investigation report. • Data and design reports for new/remedial works, if required. • Site information on river ice, debris flow, high water levels for various flood frequencies.

Step 5: Do Hydraulic Design (Water Crossing)

Figure 5.1 shows the aspects of design that may be required for a highway project. Bridges and culverts are the most frequent type of works. The element of hydraulic design that is required for all bridges and culverts is determination of conveyance capacity of the opening and estimated backwater caused by the opening for various flow rates. A bridge or culvert may require additional elements of design. These may include scour protection, river and/or debris flow analysis, and fish passage.

Other in-stream works that may be required are energy dissipators, stabilization of stream banks, and fish habitat. Occasionally, a crossing over an inland lake may be required. Each of these subjects is discussed in later sections of this chapter.

Step 6: Prepare Detail Drawings for the Water Crossing

The hydraulic engineer should review any detail drawings for the water crossing.

Step 7: Document Hydraulic Design (Water Crossing)

It is good practice to document the results of a detailed hydraulic design for review by the project engineer and for future reference. The documentation will also be needed to enable the structural/foundation engineer to design the structures. The expected output of a detail hydraulic design is shown in Table 5.4

Step 8: Deliver Detail Design and Drawings for Contract Preparation

The hydraulic engineer should be consulted, as necessary, for provisions in the contract that are applicable to the water crossing design.

Table 5.4: Expected Output of Detail Hydraulic Design

All Types of Works • Information from the preliminary design report. • Analysis/design methods used. • Major data and assumptions used. • Flow frequencies and discharges. • Analysis results. • Design notes and agreements reached between various designers of the project. **Bridges, Culverts and Lake Crossings** • Estimated water surface elevations corresponding to various flow frequencies. • Dimensioned plan, profiles and typical cross sections of crossing, abutments and piers. • Special provisions required for scour protection, if applicable. • Hydraulic details of flow transitions, if required. • Special provisions for river ice and debris flow, if required. • Special provisions for shore protection of a lake crossing, if required.	**Energy Dissipators** • Dimensioned plan, profiles and typical cross sections of energy dissipator. • Elevations of channel invert, floor of stilling basin, and water surface profiles at and after the hydraulic jump for various design flows. • Provisions required for scour protection. **Channel Stabilization** • Dimensioned plan, profiles and typical cross sections of the works area. • Key details of the stabilization works, e.g. type of cover/lining material. **Fish Habitat Hydraulic Design** • Hydraulic design details and special provisions required to assure hydraulic feasibility and sustainability of habitat.

Flow Conveyance and Backwater

Background

Before checking the bridge or culvert structure for flow conveyance and backwater, preliminary information that may be needed for the calculation are as follows.

- Conveyance: The measure of a watercourse or conduit to convey flow. Applying the Manning equation (Eq. 8.66), Conveyance, $k = A * R^{2/3} / n$
- Backwater: The increase in water surface profile elevations in a channel or conduit upstream from an obstruction to flow.

Table 5.5: Preliminary Information Checklist

Culvert	Bridge
Location and alignment of culvert Culvert profile Culvert length Culvert configuration (i.e. open footing or closed invert) Culvert material Need for multi-barrel culverts	Location and alignment of bridge structure Location and alignment of piers and abutments Preliminary span arrangement Soffit elevation
Culvert or Bridge	
Highway profile and horizontal alignment Freeboard requirements Permanent erosion control measures Relief flow requirements Fish passage flow Debris and ice flow requirements	Navigation Requirements Local stream modifications Factors related to the water crossing and the highway (i.e. skew) External constraints Minor access routes under water crossings

For information on the items noted in Table 5.5, refer to Chapter 3 in Part 1 of this manual. In addition to the preliminary information, other information pertinent to the calculation of flow conveyance and backwater are as follows.

Tailwater Level

Tailwater level, TWL, is the water level immediately downstream of a point of interest, such as a bridge or culvert site. TWL may be affected by varying water levels in a downstream lake or large river. Control for the purposes of power generation may create a large range of TWLs. Large increases in water level may occur during high flows if there is a narrow constriction downstream that creates backwater (stage-discharge relationship).

In design, the full range of possible tailwater levels must be considered for the flow rates of interest. A high TWL is usually critical for flow conveyance design. A low TWL creates higher flow velocities and is usually critical for scour and erosion analysis.

Embankment Fills Adjacent to Structures

It is advisable to limit hydrostatic head differential across an embankment to minimize the risk of significant flow through the embankment material. This phenomenon, known as *piping*, has resulted in numerous road washouts in Ontario. Soil fines are washed out which leads to collapse of the embankment. A means of reducing the likelihood of piping is to incorporate an impermeable clay seal on the upstream side of the embankment for sites where a significant head differential may exist.

The quality of material supporting and surrounding certain types of culverts, most notably SPCSPs, is an important concern for ensuring culvert durability.

Guide to Selection of Analysis Methods

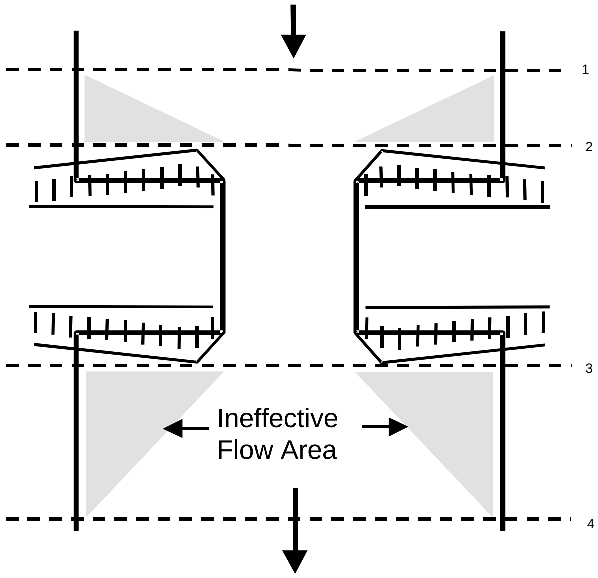
A water crossing that constricts a stream will increase the water surface elevation at the upstream side of the crossing. This backwater condition provides the hydraulic head required to overcome energy losses as the water flows through the constriction. Conveyance and backwater are, therefore, two closely related hydraulic parameters.

In determining the conveyance of a water crossing and the associated backwater, it is necessary to know which type of hydraulic method is applicable. An incorrect method may result in an erroneous design. The first step in selecting an appropriate hydraulic method is determination of the governing flow condition; that is, critical, subcritical or supercritical flow. For discussion on subcritical or supercritical flow, refer to Chapter 8.

A bridge or culvert should usually be designed to operate under subcritical flow. Special considerations may be required if supercritical flow conditions prevail. Energy Dissipators may be required to control flow (refer to the section on Energy Dissipators in this chapter).

Once subcritical flow has been confirmed, the flow conveyance and backwater should be checked. Table 5.6 provides a guide to the selection of an applicable type of method. The stream sections 1, 2, 3 and 4 are defined in Figure 5.3. Discussion of the individual methods are subsequently presented.

Figure 5.3: Site Plan



Section 1: Flow contraction begins
Section 2: Upstream face of bridge
Section 3: Downstream face of bridge
Section 4: Full re-expansion of flow is completed

Table 5.6: Guide to Method Selection

Subcritical Channel				
Crossing Type	**Section Location**	**Applicable Method**	**Where To Find Method**	**Analysis Tools**
Bridges	• Sect. 1 and above • Sect. 4 and below • Sect. 2 and 3	{Open channel flow {Open channel flow One of these three methods: • Constricted open channel flow • Pressure flow • Weir flow	Chap 8 (Eq. 8.56) This chapter (Eq 5.1) Chap 8 (Eq. 8.79) Chap 8 (Eq. 8.74)	{HEC2, {HEC- RAS {MTO Chande Model
Culverts	•Sect. 1 and up •Sect. 4 and down •Sect. 2 and 3	Same as bridges Same as bridges The controlling one of: • Inlet control • Outlet control	Orifice flow. Chap 8 (Eq. 8.79) Constricted open channel flow. See above	{HEC2, HEC-RAS, {CULVFLOW
Critical or Supercritical Channel Flow				
See Section on Energy Dissipators				

Flow through Bridges

When sizing a bridge structure, the flow through the bridge should be checked for the following cases.

Constricted Open Channel Flow can occur when channel flow is conveyed through a structure that has an associated cross-sectional flow area that is less than the cross-sectional flow area of the stream channel section that is located immediately upstream of the structure. The result can be an increase in elevation of the water surface profile upstream of the structure.

Pressure Flow can occur when the water surface profile is above the maximum soffit elevation on the upstream side of the bridge. A difference in head must exist between the water surface elevations at the upstream and downstream faces of the structure, to force the flow of water, under pressure, through the waterway opening.

Weir Flow occurs if the water surface profile is above the top of the roadway. Weir flow is sometimes called relief flow, meaning that the road profile geometry provides additional conveyance of flow in addition to that of the structure waterway opening. Whether a bridge crossing design should allow pressure flow and/or relief flow should normally be determined during the preliminary stages of design which offers the best opportunity to take all major design implications into consideration. Refer to Chapter 3 in Part 1 of this manual for a discussion on relief flow.

There are two basic ways to check for the above noted cases:

- model the crossing using an appropriate computer program; or
- use computational methods.

Computational methods can be used to determine the initial size and configuration of the waterway opening. A computer model can then be established to finalize the design. Refer to Chapter 8 for a brief overview of suitable hydraulic computer programs. The computational procedures are presented below. Examples on the computational procedures and a computer model application are presented at the end of this section.

Constricted Open Channel Flow

The head loss at a waterway opening (Bradley, 1970) may be expressed as:

$$h_T = [\, K_T\, a_2 + a_1 \{(A_2 / A_4)^2 - (A_2 / A_1)^2\}] * [V_2{}^2/2g] \tag{5.1}$$

where:

h_T = Total head loss, m
A = Cross-section area perpendicular to flow, m^2
a = Velocity head coefficient

V_2 = Velocity at entrance, m/s
K_T = Total loss coefficient
K_T = $K_b + K_p + K_e + K_s$ (5.2)
K_b = Base coefficient
K_p = Pier coefficient
K_e = Eccentricity coefficient
K_s = Skew coefficient

The number subscripts refer to the section locations. The flow direction is from Section 1 to 4. For a subcritical flow analysis, the calculation should start from the downstream end and proceed upstream.

Base Coefficient (K_b)

The base coefficient, K_b, accounts for the ratio of unimpeded flow through a cross section equivalent to a bridge opening to the total flow the opening must handle.

$$\text{Conveyance Ratio (M)} = \frac{\text{Unimpeded flow through bridge opening, m}^3\text{/s}}{\text{Total flow from opening and flood plain, m}^3\text{/s}}$$

This assumes that the flow depth is the same for both situations. Using M, the coefficient K_b may be determined. (Design Chart 5.01)

Pier Shape Coefficient (K_p)

The pier shape coefficient incorporates the effect of obstruction caused by a pier. Determine the ratio of cross section area obstructed to the total cross-section area of the bridge opening, as:

$$J = \frac{\text{Cross-Section Area of pier square to flow, m}^2}{\text{Total Area of opening square to flow, m}^2}$$

Enter Design Chart 5.02 with the estimated value of J and estimate K_p for a given pier shape. Adjust the value of K_p by multiplying with a correction for the value of M (<= 1.0).

Eccentricity Coefficient (K_e)

This variable reflects the head loss resulting from a situation where a proposed waterway opening may not be symmetrical within the natural section of a stream, resulting in a flow distribution different than that of the natural section. The eccentricity coefficient (K_e) may be estimated from Design Chart 5.03 for given values of M and e, where M is the conveyance ratio discussed above and:

$$e = (1 - Q_c/Q_a). \tag{5.3}$$

Q_c and Q_a are the overbank flows on the upstream flood plains on each side of the river beyond the projected limits of the bridge opening, with $Q_c < Q_a$. If $Q_c = Q_a$, then e = 0 and a bridge opening is placed in line with the channel.

Skew Coefficient (K_s)

Obtain the skew coefficient, K_s, by applying conveyance ratio, M; skew angle, ϕ; and bridge orientation (A or B). (Design Chart 5.04)

Velocity Head Coefficients (a_1 and a_2)

The variable a_1 in terms of the section parameters is expressed as:

$$a_1 = \Sigma(Q_n * V_n^2) / (Q * V^2) \tag{5.4}$$

where:
- Q_n = flow in a sub-section 'n', m^3/s
- V_n = flow velocity in a sub-section 'n', m/s
- Q = total flow in section, m^3/s
- V = average flow velocity for section, m/s

Estimate a_2, given a_1 and M from above: (Design Chart 5.05)

For a first approximation, h_T may be expressed as:

$$h_T = K_T * (V_2^2 / 2g) \tag{8.56}$$

Design Example 5.2 illustrates manual calculations for flow through a bridge waterway and Design Example 5.3 illustrates a HEC2 computer model application. For working details of computer tools, see the user manuals of the models, such as MTO Open Channel Model. Refer to Chapter 8 for more details.

Pressure Flow

Where a bridge soffit is fully submerged, pressure flow, Q_p, through waterway openings, may be analyzed using the following equation:

$$Q_p = C_d * A * (2gH)^{0.5} \tag{8.79}$$

where:
- Q_p = flow, m^3/s

C_d	=	coefficient of discharge for fully submerged pressure flow (Table 8.6)
A	=	Area of waterway opening, m^2
g	=	Acceleration due to gravity = 9.81 m/s^2
H	=	Difference in elevation between upstream and downstream energy grade line, m

The above equation only applies when the entire soffit of the bridge is fully inundated. Where only the upstream edge of the soffit is inundated, an *orifice* flow situation exists, requiring a different type of analysis (reference: *U.S. Army Corps of Engineers, HEC-RAS User's Manual, 1995*).

Weir (or Relief) Flow

It is desirable to have the low point of the road sag away from the structure, so that the structure is less vulnerable to washout. As discussed in Chapter 8, relief flow over an embankment may be estimated by assuming that it performs like a broad-crested weir, using the following equation:

$$Q_w = C * L * H^{1.5} \qquad (8.74)$$

where:

Q_w	=	Weir discharge, m^3/s	
C	=	Weir coefficient	(Design Chart 2.43)
L	=	Length of weir, m	
H	=	Height of upstream water surface above weir crest, m	

Figure 5.4: Typical Relief Flow Options - Bridges

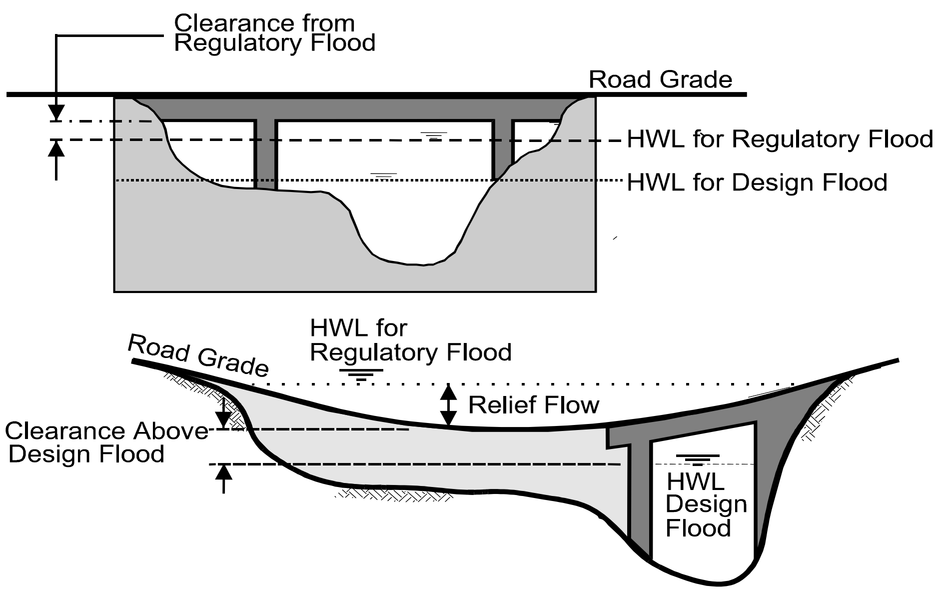
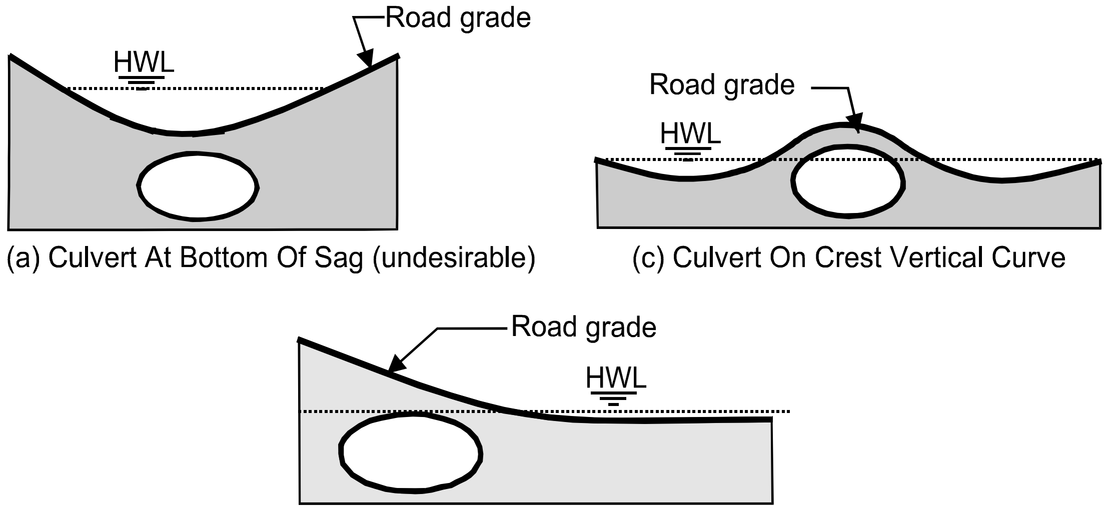


Figure 5.5: Typical Relief Flow Options - Culverts



(a) Culvert At Bottom Of Sag (undesirable)

(c) Culvert On Crest Vertical Curve

(b) Culvert At Edge Of Flood Plain (preferred)

Bridge Soffit Elevation Constraints

Often, costs of a bridge structure increase rapidly as the minimum allowable soffit elevation is increased. Therefore, for some designs, significant planning must be carried out in determining a suitable soffit elevation. In some cases, the incremental benefits of meeting all soffit elevation constraints may be less than the incremental costs. In such cases, designers should carefully document their findings and recommendations and obtain approvals to waive certain constraint(s). Refer to Chapter 3 in Part 1 of this manual for factors affecting soffit elevation. For convenience, factors affecting soffit elevation are repeated and may include:

- design HWL and freeboard requirements;
- regulatory flood conveyance, if required;
- ice or debris clearances;
- shared use constraints (i.e. pedestrians);
- navigation requirements;
- provide extra clearance for bridges that would be particularly vulnerable to side loads due to flow or impact;
- road design constraints, foundation problems, superstructure depth;
- desirable to have the soffit above the low point of the approach grade, such that relief flow is initiated before flow contacts the soffit.

Flow through Culverts

Flow through culverts is similar to flow through a bridge waterway, such that constricted flow, pressure flow and weir flow (relief flow over road embankment), may all be possible. However, for culverts, there are two types of flow that should be checked; *inlet control* and *outlet control*. Inlet control means that the conveyance capacity of the culvert is limited by the inlet capacity of the culvert. In outlet control, the outlet capacity is limiting. The flow type through any particular culvert may change with flow, Q, and associated downstream water (tailwater) levels.

Inlet control capacity is affected by inlet geometry and inlet cross-section area. Improved inlet features, such as bevels, tapers or wingwalls, can greatly improve flow capacity for culverts with flows operating under inlet control. Outlet control capacity is affected by tailwater level and barrel characteristics, such as cross-section area, surface roughness (Manning's n), longitudinal slope and length.

Whether a culvert would flow under inlet or outlet control for a particular flow rate may not be apparent from site conditions. A proper design procedure is to analyze a culvert for both inlet and outlet control conditions for a particular flow, Q. The controlling condition is the one that results in the higher headwater elevation.

Excessive flow velocities should be avoided. A culvert may be initially sized for a design flow velocity of about 2 m/s although there are many factors that a designer may consider that may affect this. Both inlet and outlet flow velocities should be considered in analysis of scour, erosion and stream stability. For an existing culvert of inadequate size, a supplementary or relief flow culvert may be added, provided that it does not cause adverse effects due to increased conveyance.

For discussion on culvert hydraulics, refer to Chapter 8.

Design Example 5.4 illustrates manual calculations for culvert design and Design Example 5.5 illustrates an application of the computer model CULVFLOW.

Improved Culvert Inlet Design

The term improved inlet is used to contrast the majority of culvert designs which use conventional shapes of culvert inlets. Improved inlets are intended to increase conveyance efficiency by reducing head loss at the culvert entrance. Improved inlets may consist of head walls and/or wingwalls. Also, side tapered and flared entrances, vertical drops and/or slope-tapered drops may be incorporated. Design Charts 5.55 to 5.59 illustrate the types of inlet improvements.

Improved inlets are suited for inlet control situations. With outlet control, the effect of an improved inlet will be reduced since tailwater levels control. Furthermore, the dominant culvert operating condition may change with the different flow conditions. For instance, a culvert that operates in inlet control for a particular flow may operate in outlet control after an improved inlet is added.

For fish bearing streams, a culvert should be designed with outlet control as a governing condition for fish passage flow because the supercritical flow prevailing under inlet control is unfavourable to migrating fish. Also, for fish passage, it is good practice to counter-sink a culvert invert for the entire length to create a more natural flow environment and tranquil velocity conditions.

Side-Tapered Inlets

A side-tapered inlet has an enlarged face area with a tapered transition to the culvert barrel. For a side-tapered inlet, there are three possible control sections: the face, the throat and the weir crest. Side-tapered box and pipe culverts can increase the flow by 25 to 40% over a conventional culvert.

The main advantages of a side-tapered inlet operating in throat control are that the flow contractions at the throat are reduced, and for a given head water elevation, more head is applied at the throat control section.

Slope-tapered Inlets

Where a fall in invert can be accommodated at a culvert inlet, slope-tapered box and pipe can be used to improve the culvert capacity. The advantage of a slope-tapered culvert over a side-tapered one is the shorter length required.

Both the face and throat are possible control sections in a slope-tapered inlet. However, since the major cost of a long culvert is the barrel portion, rather than the inlet structure, the inlet face should be designed with a greater capacity than the throat for the allowable headwater. This will ensure that flow control will be at the throat and inlet efficiency will be increased.

The mitred inlet is a variation of the slope-tapered inlet which provides an improved capacity by increasing the head on the control section.

Culvert End Treatment

In addition to the discussion above on improved inlets, there are many reasons for modifying culvert ends. Flexible culverts, such as SPCSPs may fail due to hydraulic uplift forces. This may be due to dynamic flow pressures and/or blockage of the culvert entrance by ice or debris. To overcome this, the end must be adequately stiffened or weighted down. End modifications, such as cutoff walls or aprons, may also be required to prevent scour, erosion and undermining at either end. Fill slopes may be retained by wingwalls or headwalls, thereby allowing a shorter overall culvert length. A streamlined inlet may be less prone to trapping debris.

Design Example 5.1
Conveyance of Flow through a Channel

Required

Design a trapezoidal channel to convey a given flow from an upstream watershed. A suitable bottom width and depth of flow are to be determined.

Given

- Design flow, $Q_d = 0.4\ m^3/s$
- Proposed cross section is trapezoidal with 2:1 side slopes
- Longitudinal slope, $S = 0.0012$
- Manning's $n = 0.030$ (Design Chart 2.01)
- Competent velocity of in-situ sandy-gravel bed material, $V_c = 1.0\ m/s$ (Design Chart 2.18)

Analysis

Manning's equation is:

$$Q = (A * R^{2/3} * S^{1/2}) / n \qquad (8.66)$$

Rearranging Manning's equation with Q and S (known and given) on one side, conveyance, k, may be expressed as:

$$k = Q/S^{1/2} \qquad = (A * R^{2/3}) / n$$

The required conveyance, k, is obtained as follows:

$$\begin{aligned} k &= Q/S^{1/2} \\ &= 0.4\ m^3/s / 0.0012^{1/2} \\ &= 11.5\ m^3/s \end{aligned}$$

Therefore, the channel must have minimum conveyance, k, of $11.5\ m^3/s$. The product, $A * R^{2/3}$ may be determined by rearranging the conveyance equation, since Manning's n-value is also known:

$$\begin{aligned} A * R^{2/3} &= k * n \\ &= 11.5 * 0.030 \\ &= 0.345 \end{aligned}$$

By inspection, select a suitable trapezoidal channel cross section with 2:1 side slopes that exceeds the above product of area and hydraulic radius. To analyze this, a table of values for $A * R^{2/3}$ for values of bottom width, b_e and depth, d, was developed. The proposed channel must have $[A * R^{2/3}]$ exceeding 0.345 $m^{8/3}$.

Table: Values of $A * R^{2/3}$

b_e (m)	d (m)		
	0.3	0.4	0.5
0.5	-	-	0.317
1.0	-	0.292	0.457
1.5	0.230	0.394	0.603
2.0	0.296	0.498	-
2.5	0.361	-	-

Channel cross sections capable of conveyance of design flows, $[A * R^{2/3} > 0.345\ m^{8/3}]$.
(as determined from the table above)

Table: Values of $A * R^{2/3} > 0.345\ m^{8/3}$

Width b_e (m)	design depth d (m)	$A * R^{2/3}$ ($m^{8/3}$)
1.0	0.5	0.457
1.5	0.4	0.394
2.0	0.4	0.498
2.5	0.3	0.361

All of the cross sections listed in the above table are satisfactory for conveyance for the proposed channel. The cross section finally selected for detailed design has a 2.0 m bottom width and a depth of 0.4 m, and 2:1 side slopes.

As a check, use the Manning equation to determine Q for d = 0.4 m

$$Q = (A * R^{2/3} * S^{1/2}) / n \tag{8.66}$$
$$= (1.12 * 0.30^{2/3} * 0.0012^{1/2}) / 0.03$$
$$= 0.58\ m^3/s$$

The above value of Q is somewhat greater than Q_d (= 0.4 m^3/s). Repeat the calculation with the Manning equation to determine depth, d, for the desired the conveyance capacity. The results are shown below. The value of d is 0.32 m

Table: Cross section and Flow Parameters

d (m)	A (m^2)	P (m)	R (m)	V (m/s)	Q (m^3/s)	F_r
0.3	0.78	3.34	0.234	0.438	0.34	0.26
0.4	1.12	3.78	0.296	0.513	0.58	0.26
0.32	0.84	3.43	0.246	0.453	0.38	0.26

The channel bed and side slopes would not require lining because the design velocity is less than the competent velocity, V_c (given as 1.0 m/s), of the material. The Froude number, F_r, is less than unity, indicating subcritical flow.

Design Example 5.2
Backwater Calculations for Bridge Opening

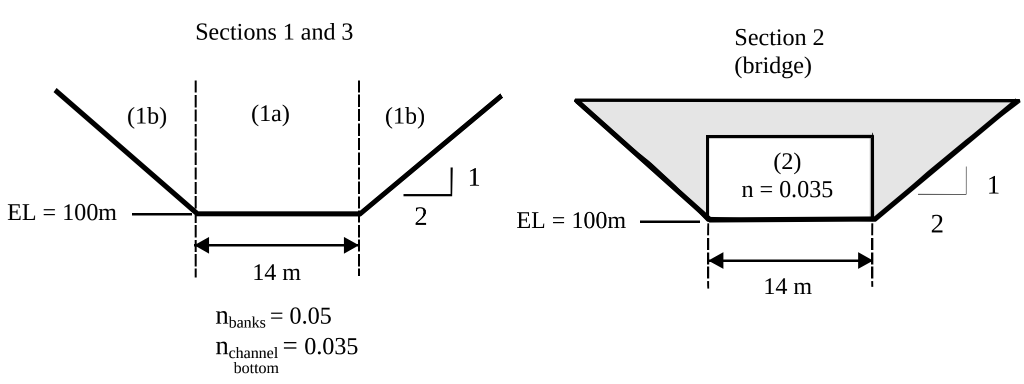
Required

Estimate backwater resulting from the waterway constriction of a proposed structure.

Given

- Design discharge, Q_{100} = 116.2 m^3
- Existing channel (see figure below)
  - straight, laterally stable
  - average invert slope 0.001 m/m
- Manning's roughness coefficient, n (Design Chart 2.01)
  - channel = 0.035
  - banks = 0.05
- zero skew
- Cross-sections
  - #1 upstream of bridge
  - #2 bridge waterway opening
  - #3 downstream of bridge
- Invert elevation at cross section 3 = 100.00 m
- Soffit elevation is clear of flow

Figure: Cross-sections



Analysis

The solution requires an iterative process.

1. Estimated Tailwater Elevation

Assuming a prismatic channel and using the above channel cross-section characteristics and invert slope, the tail water depth for the channel at cross-section #3 was estimated using the Manning equation (Eq. 8.66) as 3.13 m. The corresponding tailwater elevation, T.W. El. = 103.13 m, assuming a channel invert elevation of 100.00m at cross-section #3.

2. The following equation forms the basis for this solution:

$$h_T = [K_T.a_2 + a_1\{(A_2/A_3)^2 - (A_2/A_1)^2\}] * (V_2^2/2g) \qquad (5.1)$$

where:

h_T = total head loss (m) between cross-sections 1 and 3, m
K_T = total loss coefficient [= $K_b + K_p + K_e + K_s$] (5.2)
A = cross-section area (m^2) - subscript denotes cross-section, m^2
a = velocity head coefficient
V = velocity - subscript denotes cross-section, m/s
g = acceleration due to gravity, m/s^2

First Iteration

Try: $h_T = K_T * V_2^2/2g$ (8.56)

Assume depth = 3.13 m.

Generate cross section parameters for sections 1 and 2. Cross section 1 is subdivided into two components, a rectangular shape 14 m wide over the channel bottom and two triangular shapes over the 2:1 side slopes (banks).

Table: Cross section Parameters

Cross Section Parameters	Bridge Section (Section 2)	Upstream Section Channel (Section 1a)	Upstream Channel Banks (Section 1b)	Upstream Section Total (1a + 1b)
Manning's n	0.035	0.035	0.05	
HWL depth, d	3.13	3.13	3.13	3.13
Area, A (m^2)	43.8	43.8	19.6	63.4
Perim, P (m)	20.3	14.0	14.0	28.0
R = A/P (m)	2.16	3.13	1.40	
Conveyance, k (=A * $R^{2/3}$/n)	2091	2677	491	3168

$$M = \frac{k(\text{bridge})}{k(\text{total})} = \frac{2091}{3168} = 0.66$$

$K_b = 0.75$ (Design Chart 5.01)
with M = 0.66 and 90° wingwalls assumed

$K_p = 0$ (no piers) (Design Chart 5.02)

$K_e = 0$ (no eccentricity) (Design Chart 5.03)

$K_s = 0$ (zero skew) (Design Chart 5.04)

Therefore;

$K_T = K_b = 0.75$

Estimate the flow velocity through the bridge waterway, V_2:

$V_2 = Q / A$

$V_2 = 116.2\ m^3/s / 43.8\ m^2 = 2.65\ m/s$

For the first iteration;

$$h_T = K_T * V_2^2 / 2g \quad (8.56)$$
$$= 0.75 * (2.65\ m/s)^2 / (2 * 9.81\ m/s^2) = 0.27\ m$$

Second Iteration

Increase the assumed flow depths at sections 1 and 2, by 0.27 m to 3.40 m.

Reiterate the parameters for cross sections 1 and 2:

Table: Cross section Parameters

Cross Section Parameters	Bridge Section (Section 2)	Upstream Section Channel (Section 1a)	Upstream Channel Banks (Section 1b)	Upstream Section Total (1a + 1b)
Manning's n	0.035	0.035	0.05	
HWL depth, d	3.40	3.40	3.40	3.40
Area, A (m^2)	47.6	47.6	23.1	70.7
Perim, P (m)	20.8	14.0	15.2	29.2
R = A/P (m)	2.29	3.40	1.52	
Conveyance, k	2363	3075	611	3686

For cross section 1:
Q = 116.2 m^3/s (total flow in cross-section)
V_{n1} = 116.2 m^3/s / 70.7 m^2 = 1.64 m/s
M = k(bridge) / k(total) = 2363 / 3686 = 0.64
K_b = 0.80 (Design Chart 5.01)

where:
M = 0.64 (90^o wingwalls assumed)
$K_T = K_b = 0.80$

The flow q for each cross-section segment is proportional to its conveyance 'k'.

$$Q_n = \frac{k_n * Q}{k_{total}}$$

Q_{1a} (channel) $= k_{1a} * q / k_{total}$
= (3075 * 116.2 m^3/s)/3686
= 96.9 m^3/s

Q_{1b} (banks) $= k_{1b} * q / k_{total}$
= (611 * 116.2 m^3/s)/3686
= 19.3 m^3/s

The flow velocities for each cross-section segment are:

$V_{1a} = Q_{1a} / A_{1a}$
$= 96.9\ m^3/s / 47.6\ m^2$
$= 2.04\ m/s$

$V_{1b} = Q_{1b} / A_{1b}$
$= 19.3\ m^3/s / 23.1\ m^2$
$= 0.84\ m/s$

Determine a_1:

$$a_1 = \frac{\Sigma (Q^* V^2)}{Q * V_{n1}^2} \qquad (5.4)$$

$$= \frac{Q_{1a} * V_{1a}^2 + Q_{1b} * V_{1b}^2 ....}{Q * V_{n1}^2}$$

$$= \frac{96.9 * 2.04^2 + 19.3 * 0.84^2}{116.2 * 1.64^2}$$

$$= 1.33$$

Determine a_2:

$a_2 = 1 + M * (a_1 - 1)$ (Design Chart 5.05)
$= 1 + 0.64\ (1.33 - 1)$
$= 1.21$

Solve for h_T:

$A_3 = b * d + Z * d^2$
$= 14.0\ m * 3.13\ m + 2 * 3.13^2 = 63.4\ m^2$

$V_2 = Q_2 / A_2$
$= 116.2\ m^3/s/ 47.6\ m^2 = 2.44\ m/s$

$h_T = [K_T.a_2 + a_1\{(A_2/A_3)^2 -(A_2/A_1)^2\}] * (V_2^2/2g)$ (5.1)
$= [0.80* 1.21 + 1.33*\{(47.6/63.4)^2 -(47.6/70.7)^2\}] * (2.44^2/(2*9.81))$
$= 0.34\ m$

The total head loss, h_T, is 0.34 m between cross-sections 1 and 3.

Try a third iteration with the flow depths at sections 1 and 2 increased to 3.47 m (3.13 m + 0.34 m).

The third iteration (not shown) resulted in a total head loss, h_T, of 0.36 m. This is a slight (0.02 m) increase from the previous iteration. The procedure appears to be converging on h_T = 0.36 m and therefore, no further iterations are necessary. The upstream water level, HWL (103.13 + 0.36 =) 103.49 m.

Conclusion

The existing natural channel condition is compared to that with the proposed 14 m span bridge in place for the design flow. The existing condition was estimated using the Manning equation, assuming a prismatic channel as discussed above.

Table: Results

	Q (m^3/s)	TWL (m)	U/S HWL (m)	V (m/s)
existing	116.2	103.13	103.18	1.83
proposed	116.2	103.13	103.49	2.65

The presence of the bridge causes additional backwater in the immediate upstream vicinity of (103.49 - 103.18 =) 0.31 m above the existing channel condition with the design flow, Q_{100} = 116.2 m^3/s.

Design Example 5.3
Bridge Conveyance and Relief Flow. HEC2 Method

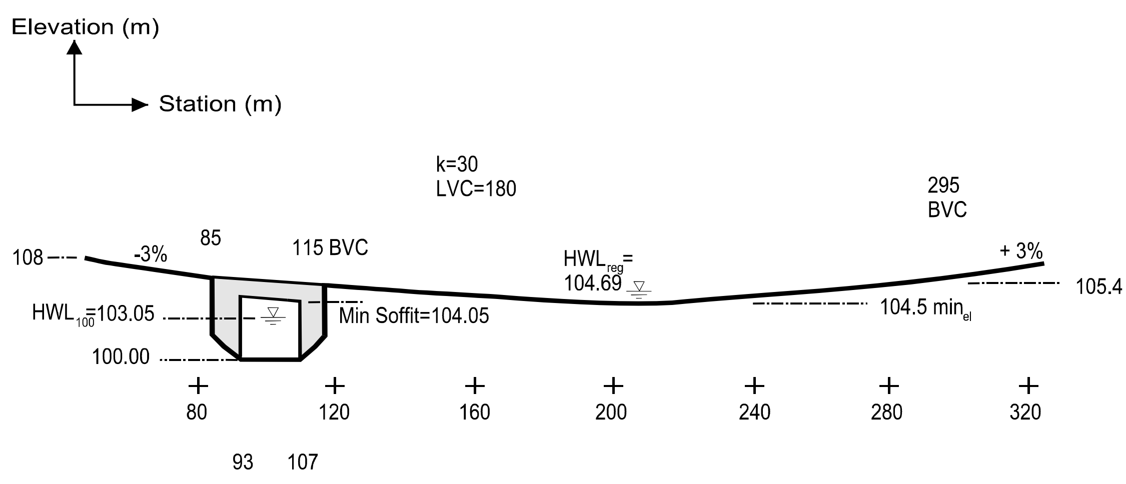
Required

Backwater analysis is required to determine the hydraulic characteristics of a proposed bridge, using HEC2 Water Surface Profiles computer model (Reference: Hydrologic Engineering Center, HEC2 manual, version 4.6.0).

Given

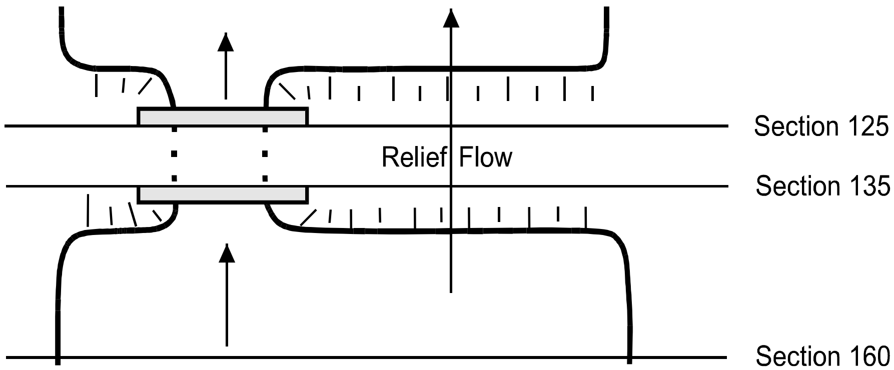
- Channel and bridge/roadway geometry, see the figure below.
- Design flow, Q_{100}, of 116.2 m^3/s, to be accommodated within the bridge waterway opening with a minimum 1.0 m clearance between the water level and minimum soffit elevation.
- Regulatory flood, Q_r of 245.7 m^3/s, to be accommodated by allowing relief flow over the roadway.

Figure: Profile Geometrics

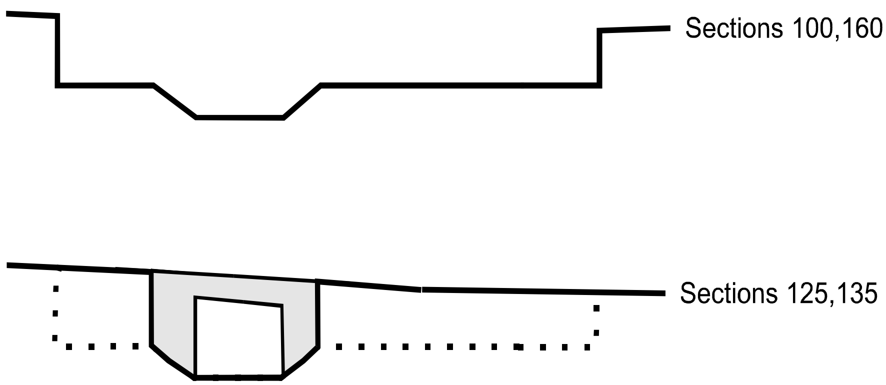


Scale: NTS

Figure: Cross-section Locations



Plan of Cross-Sections (N.T.S.)



Cross-Section Profiles (N.T.S.)

Analysis

For working details of the computer model, refer to the user manual.

The site and structure are modelled with a total of four cross sections. For subcritical flow, computations start at the downstream cross-section and proceed upstream. The two water surface profiles for the flows, Q_{100} and Q_r can be analyzed in one run of the model.

The first cross-section (station 100) represents the natural cross section of the river valley, 25 m downstream of the downstream face of the bridge. Two intermediate cross sections (stations 125 and 135) represent the downstream and upstream sides of the road and bridge profile for the full valley width. The last cross section (station 160) represents the natural cross section, 25 m upstream of the upstream side of the bridge. The following data represents the river valley with the road and bridge in place.

Table: Cross section Co-ordinates (without bridge), m

Cross section (m) Station (m)	160 upstream	135 / 125 highway location	100 downstream
80	106.55	106.45	106.35
85	106.40	106.30	106.20
85	104.10	104.00	103.90
93	100.10	100.00	99.90
107	100.10	100.00	99.90
111	102.10	102.00	101.90
115	-	104.00	-
115	-	105.40	-
145	-	104.65	-
175	-	104.20	-
205	-	104.05	-
235	-	104.20	-
265	-	104.65	-
295	102.10	-	101.90
295	105.50	105.40	105.30
300	105.65	105.55	105.45

Each of the four ground cross sections extend across the full width of the valley and cover all points that would be inundated in the event of the highest flood. All cross sections are approximately perpendicular to the river flow direction. The natural pre-existing condition is represented by cross-sections 100 and 160 and interpolation between them.

The low flow channel bottom width is approximately 14 m. The channel limits are noted in each cross-section by the left bank station (STCHL = 89 m) and the right bank station (STCHR = 111 m). The left and right bank stations must correspond to stations of that particular cross section. Any river flow outside of the bank stations is termed overbank flow.

For each cross section, Manning's n = 0.035 within the channel and, for overbank areas,n = 0.05.
(Design Chart 2.01)

A contraction coefficient of 0.3 and an expansion coefficient of 0.5 are specified for all cross-sections.
(Design Chart 2.07)

A bridge cross section is created by superimposing the bridge profile over ground cross sections. The following bridge cross section data are superimposed over cross sections 125 and 135, the downstream and upstream sides of the bridge respectively.

Table: Bridge Cross-section co-ordinates, m

Cross-section Station, m	Upper Ordinate, m (top of road / deck)	Lower Ordinate, m (bridge soffit or ground)
85	106.30	106.30
85	106.30	104.00
93	106.06	100.00
93	106.06	104.47
107	105.64	104.05
107	105.64	100.00
115	105.40	104.00
115	105.40	105.40

The cross-section area delineated by the ordinates is deemed to be occupied by the bridge structure and therefore, blocks the flow. Comparison with the ordinates for the ground cross sections 125 and 135 from the previous table shows that the bridge cross section is an exact fit, leaving a 14 m wide waterway opening with vertical walls at stations 93 and 107. The *depth* of the bridge structure becomes zero at both stations 85 and 115.

Table: Input - Natural Hydraulic Conditions (Section 100)

Return Period / Storm	Flow (m^3/s)	Water Level (m)
100 year	116.2	103.13
Regulatory	245.7	104.61

This defines the downstream flow boundary conditions from where water surface profile calculations are initiated. This is valid, provided that subcritical flow exists throughout the analysis.

Results - Hydraulic Conditions

Table: Results for flow, Q = 116.2 m^3/s

Q = 116.2 m^3/s	Natural Reach			With Bridge		
	WL, m	EGL, m	V, m/s	WL, m	EGL, m	V, m/s
X-sec 160	103.14	103.16	0.46	103.29	103.31	0.41
X-sec 135	******	******	****	103.05	103.25	1.89
X-sec 125	******	******	****	103.05	103.24	1.89
X-sec 100	103.13	103.14	0.40	103.13	103.14	0.40

Table: Results for flow, Q = 245.7 m^3/s

Q = 245.7 m^3/s	Natural Reach			With Bridge		
	WL, m	EGL, m	V, m/s	WL, m	EGL, m	V, m/s
X-sec 160	104.61	104.63	0.44	104.92	104.93	0.39
X-sec 135	******	******	****	104.57	104.84	1.75
X-sec 125	******	******	****	104.47	104.77	1.93
X-sec 100	104.61	104.62	0.41	104.61	104.62	0.41

Discussion of Results

Flow velocities for the design flow and the regulatory flow are significantly greater through the proposed waterway opening than in the natural channel (see tables above for cross sections 125 and 135). Flow velocities in the natural channel upstream (cross section 160) will be reduced with the bridge in place due to a slight increase in backwater.

The design flow, Q_{100}, is accommodated by this structure configuration. A minimum of 1.0 m clearance is provided from the high water level to the bridge soffit. The regulatory flood is accommodated through this site with relief flow over the road sag profile on approach to this structure with additional backwater of 0.31 m. The bridge superstructure is partially submerged in this flow. The increase in backwater levels is acceptable to the regulatory authorities. Design and regulatory storm flow velocities with the proposed bridge in place are in the 2 m/s range. Appropriate protective measures should be incorporated in the design if required.

The computer model HEC-RAS (Hydrologic Engineering Center - River Analysis System) is the recently released Windows™ compatible version of this program.

Design Example 5.4
Hydraulic Calculations for Box Culvert Design

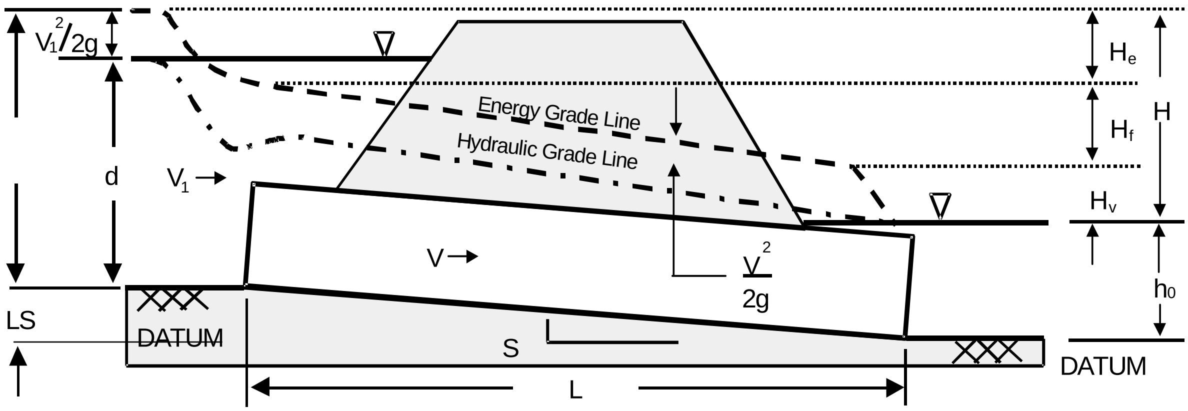
Required

Hydraulic and backwater calculations for conveyance of a box culvert.

Given

- Site Data
  - Stream channel is generally straight in the vicinity of the crossing site
  - Existing channel width varies from 3 to 5 m, average 4.0 m
  - Typical side slopes of 1.5:1
  - Crossing is at zero skew
  - Channel invert slope = 0.005 m/m
- Existing Channel
  - Design flow, Q_{100} = 30.3 m^3/s
  - Tail water depth, h_0 = 2.9 m, corresponding flow velocity = 1.25 m/s
- Proposed Culvert
  - Rectangular concrete box culvert
  - Culvert length, L = 40 m (first iteration)
  - Culvert skew = zero (skew number 90)
  - Manning's n = 0.012 for concrete (Design Chart 2.01)
  - Elevation of the stream invert at the downstream end of the culvert = 100.00 m
  - Culvert barrel to be embedded below stream invert, however, it will be filled with native material to facilitate fish passage. The embedment is not considered to contribute to the conveyance and, therefore depth of embedment, d_e, is ignored in computation.
  - Culvert longitudinal slope, S = 0.005 (to match channel invert slope)
  - Number of culvert barrels, N = 1
- Additional Requirements
  - The water surface profile in the culvert should not be higher than the original stream profile under the design flow situation.
  - Maximum flow velocity, V, in natural channel = 2 m/s

Figure: Culvert Profile



Analysis

Trial Size

Estimate the initial trial size:

It is desirable for design flow velocities to be less than 2.0 m/s. With a tailwater depth of 2.9 m, the flow per unit width would be (2.0 m/s * 2.9 m =) 5.8 m^2/s or m^3/s per m of width. For the design flow of 30.3 m^3/s, the initial trial width would be (30.3 m^3/s / 5.8 m^2/s =) 5.2 m. Round upward to 6.0 m width.

Waterway depth, D: The culvert roof should be clear of the design flow tail water level, h_0 = 2.9 m. Therefore, the culvert waterway depth, D, should be a minimum of 2.9 m; say 3.0 m.

For hydraulic analysis, the embedment is ignored, such that the culvert invert matches the stream bed profile in the vicinity with a channel slope of 0.005.

Design Procedure

The design procedure involves analysis of both the inlet and outlet control cases. The case which results in the higher headwater level for a particular flow rate is controlling.

The following equation is used with the assumption that velocity head ($V^2/2g$) for the flow, both upstream and downstream of the culvert, is not significant. Velocity head is taken as zero, such that the water surface and energy grade line are coincident away from the immediate vicinity of the culvert. The velocity head may be ignored unless the approach velocity is exceptionally high.

$$HW = H + h_0 - LS \quad (8.86)$$

where:

HW = Allowable head water depth, m
H = total head loss through culvert (entrance loss + friction loss + velocity head of flow in culvert), m
h_0 = Tail water depth, m
LS = length * slope (change in invert elevation of culvert from upstream end to downstream end), m

For discussion on Eq. 8.86, refer to Chapter 8 and the figure above.

Area of trial culvert cross-section,

A = B * D
= 6.0 * 3.0 m
= 18 m^2
(D >= HW, such that the design flow water level is clear of the soffit)

Inlet Control Calculation, calculate headwater depth, HW

Headwater/culvert Depth (HW/D), given:

- Q = 30.3 m^3/s,
- number of barrels, N = 1,
- culvert depth, D = 3.0 m;

HW/D = 0.78 (Design Charts 2.31 to 2.33 and 5.39 to 5.45)
HW = (HW/D) * D
HW = 0.78 * 3.0 m = 2.3 m

Outlet Control Calculation

Entrance Loss Coefficient, assuming 90^o square edge at entrance, $k_e = 0.5$ (Design Chart 2.08)

Head, H, given:

- Q = 30.3 m^3/s,
- number of barrels, N = 1,
- culvert depth, D = 3.0 m;

H < 0.1 m, take as 0.1 m (Design Charts 2.34 to 2.36 and 5.46 to 5.49)

Velocity head and friction losses (K_f , K_{exit}) within the culvert barrel are accounted for by the design charts.

Calculation of critical depth, d_c, is required so that equivalent hydraulic depth may be calculated below.
Critical Depth, d_c, given:

- Q = 30.3 m^3/s,

- number of barrels, N = 1,
- culvert width, B = 6.0 m;

d_c = 1.4 m (Design Charts 5.50 to 5.54)

Determine the equivalent hydraulic depth, $(d_c + D)/2$, the average of critical depth, d_c, and culvert depth, D. The equivalent hydraulic depth is located at mid height between the tailwater depth and the top-of-culvert. Critical depth, d_c, must be less than culvert depth, D.
$(d_c + D)/2 = (1.4 + 3.0)/2 = 2.2$ m

Tailwater depth in culvert, h_0, the depth of flow in the culvert at the outlet end
h_0 = 2.9 m (given).

Analysis of outlet control flow is dependent on tailwater depth, h_0. If $(d_c + D)/2$ is greater than h_0, then h_0 should be taken as $(d_c + D)/2$. For all cases of outlet control, the tailwater depth, h_0, is taken as the greater of actual tailwater depth, h_0 or the equivalent hydraulic depth, $(d_c + D)/2$.

h_0 is greater than $(d_c + D)/2$, and therefore h_0 is unchanged:
h_0 = 2.9 m

Headwater depth, HW (adjust tailwater depth, h_0, for total head loss through culvert and culvert invert slope).

$$\begin{aligned} HW &= H + h_0 - (LS) \\ &= 0.1 \text{ m} + 2.9 \text{ m} - (40 \text{ m} * 0.005) \\ &= 2.8 \text{ m} \end{aligned}$$

Head Water Depth is the larger value of HW from the inlet control (HW = 2.3 m) or outlet control (HW = 2.8 m) calculations. The HW value from the outlet control calculation is greater and therefore, the culvert would operate under outlet control for this flow rate.
HW = 2.8 m

Flow Velocity at outlet,

$$\begin{aligned} V_o &= Q / A \\ &= Q / (B * h_0) \\ &= 30.3 \text{ m}^3/\text{s} / (6 \text{ m} * 2.9 \text{ m}) = 1.75 \text{ m/s} \end{aligned}$$

The inlet flow velocity is similarly calculated using the inlet depth of flow, HW.

Final Culvert Selection

The design flow HW is 2.8 m, leaving 0.2 m clearance to the soffit. Accordingly, a concrete box culvert with a waterway opening of 6.0 m width and 3.0 m height not including embedment is satisfactory. The design hydraulic conditions for this opening are listed below:

Table: Results of Hydraulic Analysis of Proposed Opening

Return Period (years)	Flow (m^3/s)	h_0 (m)	HW (m)	V_i Inlet (m/s)	V_o Outlet (m/s)	Flow Control
100	30.3	2.9	2.8	1.80	1.75	Outlet

Conclusion

Design details of wing walls, cutoffs, bevels and scour protection should be provided. These aspects of design are not covered in this example. The design charts allow for variances in configuration.

The proposed waterway opening of 6.0 m width * 3.0 m height is adequate for the design flow. The maximum headwater depth is 2.8 m, below the maximum allowable depth of 2.9 m. Inlet and outlet flow velocities are both below the maximum allowable 2.0 m/s. The flow is 0.2 m clear of the soffit under the design flow condition. The design procedure may be reiterated with a smaller width, B, to determine whether a smaller size would be adequate (iteration not provided).

The hydraulic effects of the regional storm flow should be checked to ensure that flood levels are not affected beyond acceptable limits. Fish passage requirements should also be addressed.

Culvert Size	Span 6.0 m, Rise 3.0 m
Length	40 m
Upstream end Invert Elev.	100.2 m
Downstream end Invert Elev.	100.0 m
Tail Water Elev.	102.9 m
Head Water Elev.	103.0 m
Inlet Velocity	1.80 m/s
Outlet Velocity	1.75 m/s
Upstream end Soffit Elev.	103.2 m

The headwater level is calculated to be at elevation 103.0 m, 0.2 m below the upstream end soffit elevation.

Design Example 5.5
Estimating Flow Conveyance with CULVFLOW

Required

Evaluate several culvert configurations as part of a preliminary design, using CULVFLOW, a hydraulic computer model for culvert analysis.

Given

- Mean longitudinal channel slope = 0.003
- Channel bed elevation = 100.00 m (at upstream end of culvert)
- Channel bottom width = 10 m
- Side slopes 2:1
- Manning's roughness coefficient, n = 0.030 (Design Chart 2.01)

Table: Existing Conditions

Flow Event	Flow (m^3/s)	W.L. Elev (m)	Flow V (m/s)
100 year Regulatory	116.2 245.7	103.04 104.52	1.90 2.36

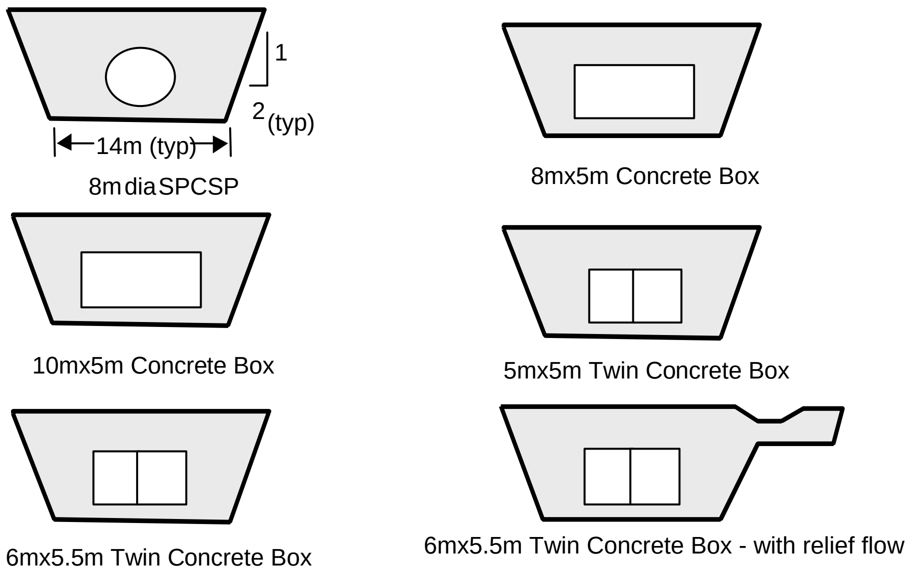
Table: Design Criteria

Flow Event	Max. Flow V (m/s)	Max. Headwater Elev. (m)
100 year	3	104.5
Regulatory	n/a	105.8

Analysis

This design example investigates six different culvert configurations. In each case, the culvert is placed on a slope of 0.003 m/m and embedded below the stream bed by 0.6 m. The six configuration cross-sections are shown below.

Figure: Trial Culvert Cross-sections, NTS



Trial Waterway Opening

The table below shows the input file for a typical run of CULVFLOW for the first trial, the 8.0 m diameter SPCSP.

Culvert inlet invert elevation:	100.00 m
Culvert outlet invert elevation	99.91 m
Culvert length	30 m
Culvert diameter	8.0 m
Culvert shape	round
Manning's roughness coefficient	0.032
Entrance loss coefficient	0.9
Flow, Q	116.2 m^3/s
Tailwater elevation	103.04 m

Results

The results of the CULVFLOW runs are listed below.

Table: (1) SPCSP: 8.0 m diameter, 30 m length, n = 0.032

Flow (m^3/s)	T.W.El (m)	H.W.El (m)	Outlet V (m/s)	Backwater Incr (m)
116.2	103.04	106.16	5.24	3.03
245.7	104.52	109.66	6.87	5.05

Table: (2) Concrete Box: 8 m wide * 5 m height, 30 m length, n = 0.015

Flow (m^3/s)	T.W.El (m)	H.W.El (m)	Outlet V (m/s)	Backwater Incr (m)
116.2	103.04	105.39	4.93	2.26
245.7	104.52	108.78	6.74	4.17

Table: (3) Concrete Box: 10 m wide * 5 m height, 30 m length, n = 0.015

Flow (m^3/s)	T.W.El (m)	H.W.El (m)	Outlet V (m/s)	Backwater Incr (m)
116.2	103.04	104.55	3.95	1.42
245.7	104.52	107.48	5.53	2.87

Table: (4) Twin Concrete Box: 2 * 5 m wide * 5 m height, 30 m length, n = 0.015

Flow (m^3/s)	T.W.El (m)	H.W.El (m)	Outlet V (m/s)	Backwater Incr (m)
116.2	103.04	104.71	3.83	1.58
245.7	104.52	107.67	5.42	3.06

Table: (5) Twin Concrete Box: 2 * 6 m wide * 5.5 m height, 41 m length, n = 0.015

Flow (m^3/s)	T.W.El (m)	H.W.El (m)	Outlet V (m/s)	Backwater Incr (m)
116.2 245.7	103.00 104.49	104.20 106.80	3.21 4.52	1.08 2.19

Table: (6) Twin Concrete Box: 2 * 6 m wide * 5.5 m height, 41 m length with relief flow over road, n = 0.015

Flow (m^3/s)	T.W.El (m)	H.W.El (m)	Outlet V (m/s)	Backwater Incr (m)
116.2 * 245.7	103.00 104.49	104.20 105.57	3.21 3.71	1.08 0.96

* Based on both, culvert flow and relief flow (50 m^3/s) over the roadway sag.

Flow split: relief flow = 50 m^3/s (calculated as weir flow using Equation 8.74), and culvert flow (= 245.7 m^3/s - 50 m^3/s) = 195.7 m^3/s.

Discussion

The results for the 6.0 m SPCSP show that the backwater for the 100-year flow is in the order of 3 m which is excessive. The flow velocity at the culvert outlet is up to 6.87 m/s for the regulatory storm. The hydraulic conditions are severe, compared to the existing natural conditions and, therefore, the opening is not suitable.

The results for the 8 m * 5 m and 10 m * 5 m concrete box culverts, though less severe than the SPCSP, are still beyond the acceptable range due to high water levels and flow velocities. Subsequent to the review of these two options, larger twin-cell concrete box culverts are evaluated.

The last option tried looks promising. Considering that passing the regulatory flow through the culvert opening will still cause unacceptable backwater levels, try the relief flow option to pass the regulatory flow while lesser flows will be accommodated by the proposed culvert opening.

Relief flow is provided by lowering the low point of the road sag to elevation 104.8 m and resulting in the road sag acting as a broad crested weir in the event of the regulatory flow. The road sag provides a profile similar to a parabola. To simplify calculations, an approximately equivalent rectangle is assumed to replace the parabolic shape of the road sag profile. Determine the flow over the road sag by trial and error. In each trial, the flow over the weir (road sag) is subtracted from the total flow. The remaining flow passes through the waterway opening and is analyzed with CULVFLOW.

The weir flow is calculated by

$$Q = C * L * H^{1.5} \quad (8.74)$$

where:

C = 1.5 (broad crested weir)
L = 150 m (length of inundated road sag)
H = 0.37 m (one half of maximum flow depth over sag)

$$Q = 1.5 * 150 * 0.37^{1.5}$$
$$= 50 \text{ m}^3/\text{s}$$

The weir flow 50 m^3/s subtracted from the total regulatory storm flow of 245.7 m^3/s, resulted in the flow of 195.7 m^3/s through the culvert at El. 105.57.

Conclusion

Table: Results

	Design Criteria (allowable)	Option 6 (estimated)
100 yr flow V (m/s)	3.0	3.2
100 yr HWL El. (m)	104.5	104.2
Regulatory flow V (m/s)	n/a	3.7
Regulatory HWL (m)	105.8	105.6

Option 6, the twin cell concrete box culvert appears satisfactory for this site. Relief flow is provided over the roadway for the Regulatory storm flow which provides backwater levels acceptable to the regulatory authorities.

Scour

Channel Scour

Many streams in Ontario are formed in semi-cohesive or non-cohesive soils which may be eroded. The predominant soil types found in stream channels are sand, silt and clay.

Scour in a stream channel is the lowering and/or widening of the stream bed due to erosive forces exerted by flowing water. Channel scour is an important consideration in the design of water crossings and stream channels as scour may undermine the foundations of a structure, possibly leading to its failure. There have been documented failures of structures as a result of scour. Channel scour design requirements are incorporated in AASHTO publications and the Ontario Highway Bridge Design Code (OHBDC), 1991.

The purpose of this Section is to provide an understanding of scour, scour estimating methods and their applications. For discussion on how stream behaviour interacts with sediment transport of a stream, refer to Chapter 9.

Flowing water in a stream channel exerts force in the direction of flow on the channel boundary surface. If the boundary force due to flow exceeds the resisting force of the boundary material, bed material particles are dislodged. A particle then moves downstream to a location where flow velocity slows enough to allow it to settle out. Scour and erosion processes occur in this manner.

Channel scour may occur in several forms as discussed below.

Natural Scour

A stream channel goes through progressive bank and bed scour over time due to naturally occurring flows and stream processes, resulting in sediment transport and channel adjustment. The occurrence of scour in the absence of any structural interference is commonly referred to as *natural scour*.

A stream channel may not have fully "matured" into a stable configuration and may still be adjusting to achieve an equilibrium configuration. Such a channel may continue to experience scour and deposition, alternating with changes in flow.

General Scour

The local lowering of a channel bed in the vicinity of a structure waterway opening is called *general scour*. It differs from natural scour because it is caused by the constriction of a structure waterway opening.

A waterway opening for a crossing may cause or change scour in several ways.

- Naturally occurring scour in a stream channel may be aggravated due to increased concentration of flow through the opening. A channel not exhibiting any natural scour may become scour-prone, if the flow is constricted.

- Scour at a culvert may be aggravated due to concentrated flows at its outlet.

- Scour potential may be reduced by improving the shape and skew of a crossing. Protection measures, such as rip rap may be utilized.

The magnitude of general scour is influenced by the location, shape, size and skew of the waterway. Figures 5.6 and 5.7 illustrate general scour.

Figure 5.6: Channel Scour - Longitudinal View

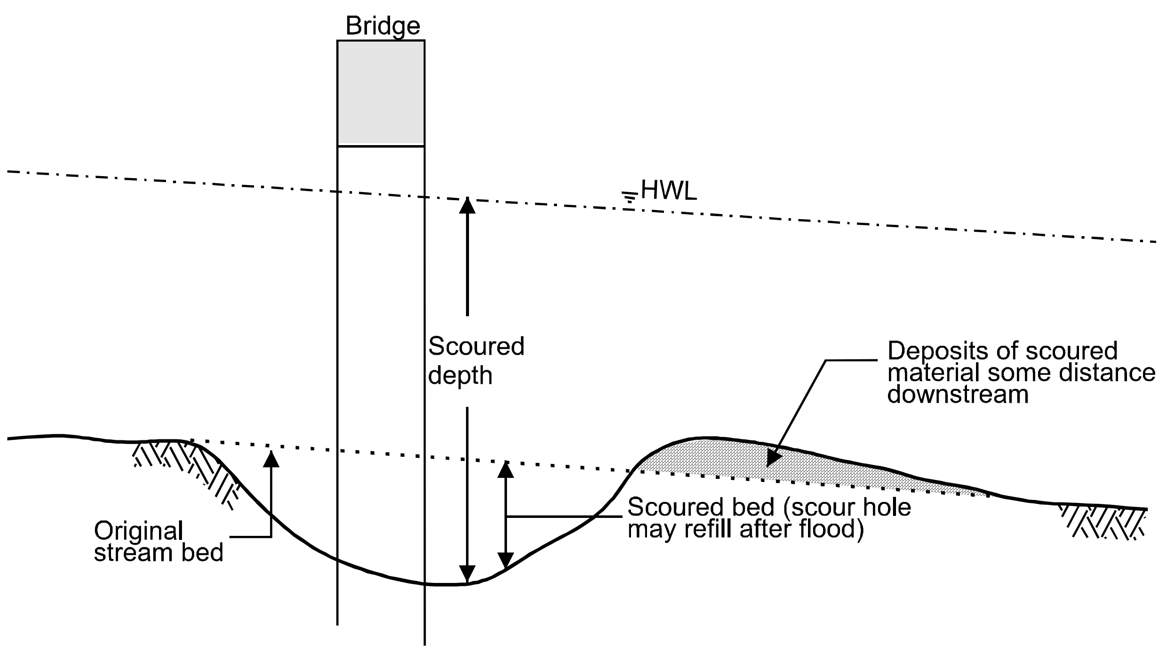


Figure 5.7: Cross Section View of Scour in a Shifting Channel

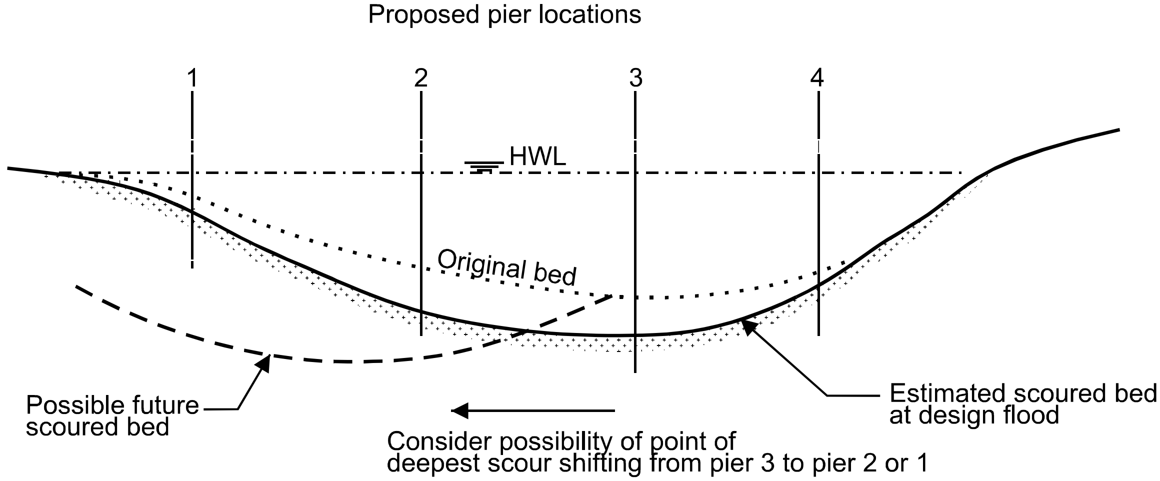
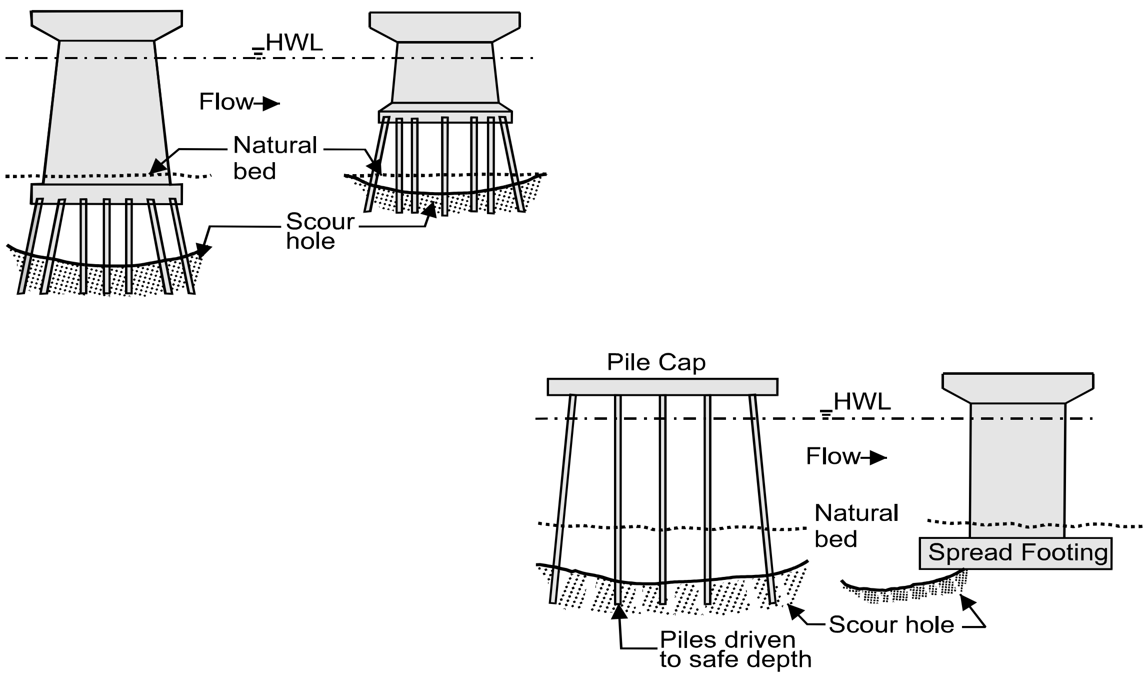


Figure 5.8: Local Scour at a Bridge Pier

Local Scour

The lowering of a stream channel bed may also occur due to flow distortion around obstructions, such as piers or groynes. Bed degradation that is generally localized around an obstruction is called *local scour* (Figure 5.9). The depth of local scour is in addition to the depths of natural or general scour in the vicinity.

Figure 5.9: Local Scour at a Bridge Pier



Factors Affecting Channel Scour

Erosion and stability of a stream depend on the balance of the natural forces acting on the stream. The following factors may increase or offset this natural balance.

Flow Discharge

A greater flow discharge may result in increasing channel velocity and shear force, therefore increasing its capacity to erode. The most critical flow case for scour often occurs with the bank-full flow condition.

Stream Width, Depth and Slope

A change in flow width, depth or slope may affect channel flow velocity. Scour potential would be similarly affected. The channel may not have reached an equilibrium condition and, therefore, ongoing aggradation or degradation may be occurring. Also, the channel may be unstable laterally, such that channel shifting or meandering is occurring.

Changes may occur that will affect flow at the site of interest. Controls, such as dams constructed both upstream or downstream may affect flow rates and water levels. Artificial deepening or widening of the channel may also affect hydraulics.

Sediment Carrying Capacity

The sediment carrying capacity of a stream is directly related to the shear stress (force per unit area), which occurs between the flow and the channel surfaces. Shear stress varies with changes in flow depth and velocity. If the sediment carrying capacity of a stream is greater than the amount of sediment actually being carried, erosion of the channel boundary may occur. Conversely, if the sediment load is greater than the carrying capacity, deposition may occur.

Bed Material

Bed material characteristics and its gradation influence channel scour. If shear stress acting on a channel surface is sufficient to overcome gravitation, inertia and cohesion of the sediment, scour will occur. Conversely if the flow velocity reduces, such that the shear stress is no longer adequate to overcome friction, inertia and gravity, deposition will occur.

Shear resistance of surface material on a slope will be reduced because a component of gravity acts to dislodge a particle to roll it down-slope. Theoretically, assuming no cohesion, a slope at an angle equal to the angle of repose of the material would have no resistance to shear stresses and, therefore, no resistance to scour.

Constriction in Channel Opening

A constriction (narrow opening) in a stream channel may occur naturally on one or both sides due to changing land forms. A constriction may also be created due to a berm, a dike or abutments of a bridge or culvert. A constriction may result in local increase in flow concentration and velocities. Consequently, shear forces acting in the channel may locally increase and scour may increase.

Obstruction in Channel Opening

Piers, groynes and cofferdams may obstruct the flow. Also, the waterway area may be significantly reduced if blocked by ice or debris. In addition to reducing flow conveyance capacity, an obstruction may aggravate scour by concentrating flow and increasing flow velocity. Poor alignment of piers and abutments may cause strong eddy currents to form that may significantly aggravate local scour. Figure 5.8 shows a typical bridge pier where scour may occur.

Field Measurements

Scour may be determined from field measurements such as probing affected areas, surveying the stream bed and underwater sounding. Geotechnical investigations may also yield helpful information to aid in scour prediction. These measurements may be used to determine the design scour conditions.

Despite extensive research and development, methods for measuring scour in the field are not exact. Recent developmental efforts in remote sensing and radar technology may result in better measurement tools in the future. Ground Penetrating Radar (GPR) can be used for some geotechnical investigations and may be applied to evaluate an existing or potential scour problem.

Ideally, scour measurements should be carried out at or close to the time of the flood peak. However, it is not always practical or safe to do so. Therefore, a common practice is to carry out scour measurements after floods. The difficulty with this practice is that maximum scour may not be measured due to subsequent in-filling of a scour hole or shifting of a stream channel. Rivers and streams formed in sand and gravel beds are typical examples of this type of occurrence. Where the history of flooding and scour can be traced by consulting records, archives or local residents, the reliability of the predicted results may be increased. However if flow records are unknown, scour measurements may not yield reliable predictions.

Estimating General/Natural Scour

Although predicting general or natural scour is an important aspect of design and is routinely carried out for the design of water crossings, the available estimating methods are empirical and built on experience and judgment. This is in spite of the fact that a significant amount of research has been conducted in recent decades. The methods are not generalized or universally applicable. When using or selecting from the four methods listed below, consider the following aspects:

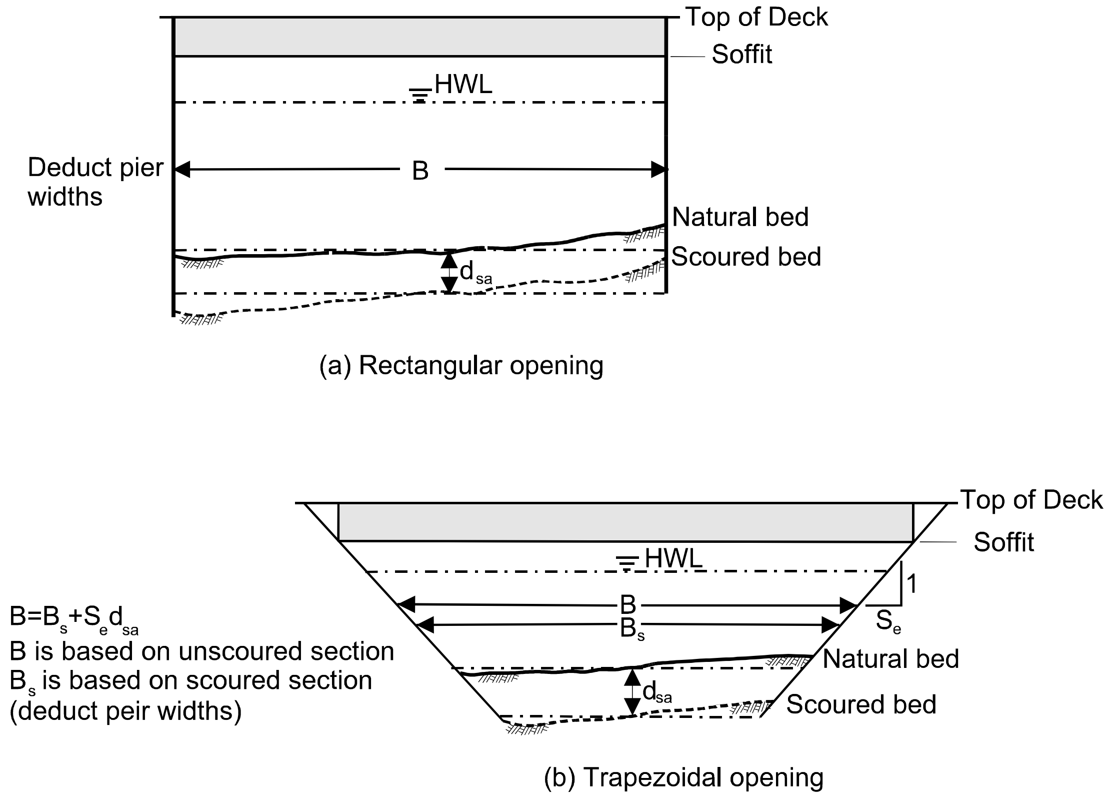
- For any particular application, various methods should be considered in regards to the site characteristics.
- The limitations of each method should be reviewed. If a particular method is not suited to site conditions, it should not be used.
- Scour depths resulting from any analysis should be compared with soil stratigraphy at that

depth, including relative compaction, to verify that the initial assumptions regarding soil properties are valid.

- Selection of the method most suited to a particular site requires experience and judgment. As illustrated in Design Example 5.6, the results from various methods may vary. Therefore, caution and judgment should be used in their application. Experienced hydrotechnical experts may be able to provide advice regarding particular applications.
- Interpretation of soils data may be sought from a geotechnical expert.

The cross section area of scour may be distributed as shown in Figure 5.10. For relatively straight channel reaches, a parabolic shaped distribution is typically used. For more sharply curved reaches, a triangular shaped distribution is used. The triangular shaped cross section distribution is more critical because it results in greater scour depths.

Figure 5.10: Cross Section Distribution of General Scour



Four methods to analyze general scour are described below. The application of each of these methods is illustrated in Design Example 5.6.

- Competent Velocity Method,
- Mean Velocity Method,
- Regime Method,
- Laursen Method.

Competent Velocity Method

This method has been widely applied in Ontario for the design of water crossings. The method is based on the assumption that a stream channel will continue to experience scour for a specified flow condition until the cross section has been enlarged such that the flow velocity is reduced to the competent velocity of the bed material. The competent velocity of the material is defined as the velocity of flow which, if exceeded, will result in the movement (erosion) of material.

Mean Velocity Method

This method (Neill, 1973) uses the mean flow velocity as a measure to compare the scour condition with the typical bank-full flow condition of a natural stream channel. The assumption is that the existing stream channel under a bank-full flow condition, has matured and stabilized over the years and is in regime. Consequently, this method would not apply to streams which are progressively degrading, aggrading or self-adjusting. Due to this inherent assumption, the method should be used with caution or as a check method only.

Regime Method Using Field Data

This method is based on the work by Blench for stream channels formed in sand and gravel strata (Neill, 1973). The method is based on using the site specific characteristics of a stream channel and therefore relies on field information.

Two flow conditions are analyzed, the bank-full flow and the design flow. Using Manning's equation and field information, such as representative cross sections, longitudinal slope, S, and Manning's roughness coefficient, n, determine the bank-full flow, Q_b. In a similar manner, Manning's equation is used to solve for the water level that results from the design (or check) flow, Q_f. Computer programs, such as CHANDE or HEC-RAS may be used to facilitate this.

The inherent assumption is that the stream channel is in regime for the bank-full flow condition. The in-regime flow depth is calculated for the design flow. For the design flow, the actual depth is subtracted from the calculated in regime depth resulting in the predicted depth of scour.

Laursen Method

The Laursen (1960) Equation may be applied in the following types of situations.

- The flood plain is narrow and does not carry flow conveyance and the proposed bridge opening closely matches the existing channel. This situation is typical of hilly terrain in gorge-type settings.

- The proposed opening is set back from the main channel and is to accommodate overbank flows. This situation is typical of the majority of bridge crossing situations. The size of the opening should be dependent on the flow to be conveyed and backwater which can be tolerated.

- The proposed opening crosses the full width of the stream channel and flood plain. This situation is generally applicable where the crossing location is sensitive and the integrity of the flood plain must be preserved.

This method compares the flow through two adjacent cross-sections. One cross section represents the structure waterway opening and the other represents a typical unobstructed upstream cross section. The equation solves for scour depth using flow rates, Q; main channel depth, d and cross section waterway widths.

Scour at Culvert Outlets

Scour computations at culvert outlets may be carried out by using one or more of the above methods. It is the engineer's responsibility to determine the suitability of any method for any particular application.

At a culvert outlet, the channel cross section usually diverges in the downstream direction until it assumes the cross section of the existing channel. As the cross section widens, flow width will increase, thereby reducing discharge intensity, q (m^3/s per metre of width or m^2/s). Therefore, the maximum channel scour will tend to occur near the outlet and this should be accounted for in culvert design.

Scour at a Groyne (Spur Dike)

Groynes or spur dikes are constructed to guide and deflect flow away from areas susceptible to scour in channels. Since a groyne obstructs the stream flow, scour at its nose may occur.

A tee-head spur may behave similarly to a guide bank except at the ends where a rapid change in curvature due to its shape, may aggravate scour. Away from both ends, scour is generally caused due to a constriction in the waterway width. The maximum scour will likely occur at the ends of a tee-head. The behaviour of a round nose groyne is similar, but reduced in lateral extent due to its round shape.

Scour susceptibility should be accounted for in design.

Estimating Local Scour

Methods for estimating local scour caused by piers are mainly empirical, based on laboratory flume studies in Europe, Australia and the U.S.A. with limited field testing (Breusers 1963, Larras 1965, Shen 1970, Melville and Sutherland 1988). Most of the available methods relate the depth of scour below the channel bed to the width, shape and the skew of a pier with no explicit account for flow. These methods are not universally applicable and results from these methods may vary. Therefore, they should be used with caution. Descriptions of these methods is provided below and their application is shown in Design Example 5.6. When selecting various methods for use, the following aspects should be considered.

- Local scour may be substantially increased by ice or debris accumulations around a pier. Therefore, refined estimates of local scour are not usually warranted in most Ontario situations. The *Ontario Highway Bridge Design Code* addresses local scour and specifies minimum requirements for design.

- The applicability of any particular method depends on site-specific conditions. Selection of the method most suited to a particular site requires experience and judgment. As illustrated in Design Example 5.6, results may vary greatly between different methods.

- Soil characteristics at estimated scour depths and below should be compared against those assumed for the analysis. Soil characteristics, such as compaction or soil type, vary with depth and may significantly affect scour susceptibility. For interpretation of soils data, advice may be sought from a geotechnical expert.

RTAC Guide to Bridge Hydraulics (1973) Method

Design Chart 5.08 shows the scour depth relationships based on the work by Larras (1965), Breusers (1963) and Neill (1973) and has been extensively used in Ontario. This method has been adopted by the *Ontario Highway Bridge Design Code* (1991).

Colorado State University (1977) Method

This method is reported in the AASHTO (American Association of State Highway and Transportation Officials) *Model Drainage Manual* (1991), and is expressed as:

$$y_{ls} = 2.0 * k_1 * k_2 * (a/y_1)^{0.65} * F_{r1}^{0.43} \qquad (5.6)$$

where:

y_{ls}	=	Depth of local scour at a pier below HWL, m	
k_1	=	Correction factor for pier shape	(Design Chart 5.09)
k_2	=	Correction factor for angle of attack	(Design Chart 5.09)
y_1	=	Flow depth below HWL just upstream of pier, m	
F_{r1}	=	Froude number at point 1 @ y_1	
a	=	Width of pier, m	

Melville and Sutherland (1988) Method

This method, proposed by Melville and Sutherland (1988), has gained acceptability among practitioners in recent years. In concept, the method attempts to relate scour depth due to flow with bed material characteristics. The effects of *bed paving* are considered to occur over time as a good gradation of bed material forms an armour layer through a *sorting* effect. Through progression of scour around a pier, a threshold depth and bed layer of well graded material will form, beyond which scour will not theoretically proceed, provided that the flow velocity does not exceed the competent velocity of the armour layer. According to this method, local scour around a pier is expressed as:

$$d_{sp} / b_p = k_i * k_y * k_d * k_\sigma * k_s * k_\alpha \qquad (5.7)$$

where:

d_{sp}	=	equilibrium depth of local scour, m
b_p	=	pier width normal to flow, m - usually given or assumed

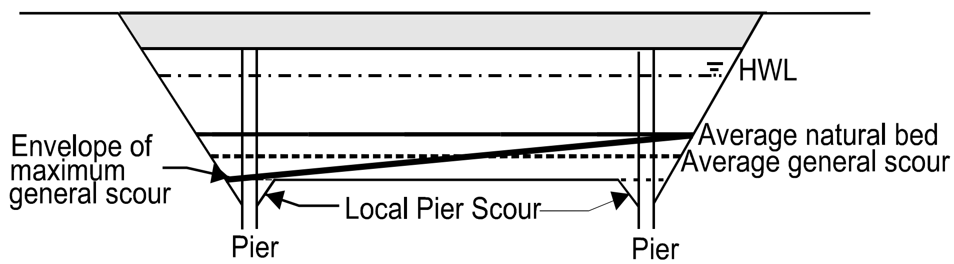
The constants account for the effects of flow intensity (flow per unit width), k_i, flow depth, k_y, sediment size, k_d, sediment gradation, k_σ, pier shape, k_s, and pier alignment, k_α. For further details on this method and its development, reference: Design Method for Local Scour at Bridge Piers, by B.W. Melville and A.J. Sutherland, *Journal of Hydraulic Engineering*, Vol. 114, No. 10, October 1988.

Using Scour Estimates in Designs of Water Crossings

- An estimated scour elevation should be compared with foundation investigation findings, if available. If a non-erosive layer, such as hard rock, exists at a shallower depth than the estimated scour elevation, examine whether it is safe to assume that this layer may act as a limit to scour.
- Structure scour protection requirements should be designed for the design flood and modified, if necessary, to ensure that structural failure will not occur as a result of the check flood. (Reference: Ontario Highway Bridge Design Code, 1991). Reliance on protection measures, such as gabions or riprap for the full lifetime of the structure is not recommended.
- If scour estimates are considered excessive, the size of the waterway opening may be increased to reduce channel flow velocities and, hence, reduce scour potential. This process may require several iterations before a desired balance is achieved.

- For a bridge opening, embankment protection should be provided to resist scour to as low as the estimated general scour elevation.
- Scour estimates should be considered in designing a pier footing. The combined effects of general and local scour on pier footings is shown in Figure 5.11. The top of a spread footing should be placed at or below the maximum general scour elevation. In order to dampen eddies and the resulting local scour, the width of the footing all around the pier shaft should be a minimum of twice the pier width normal to the flow.

Figure 5.11: Scour - Footing Interaction



- Pile foundations are generally much more able to withstand scour than spread footing foundations. It may be cost effective to specify piles and base the footing higher as well as provide a smaller waterway opening. Pile stability should be checked with the streambed at its ultimate scoured elevation (OHBDC, 1991).
- Depths of scour shall be estimated for all structures with erodible inverts at waterway crossings. Check flood flow depths shall be those of ice-free conditions and be based on the lowest downstream water level likely to coincide with such a flood. (OHBDC, 1991).
- If scour protection is required around a pier, the protection material should be placed to a depth equal to the estimated pier scour depth.
- Cut-off walls should be provided for culverts with concrete inverts in consideration of predicted scour and degradation.
- General scour may be increased by ice jams obstructing the waterway and causing a concentration of flow.

Design Example 5.6
Estimating Channel Scour

Required

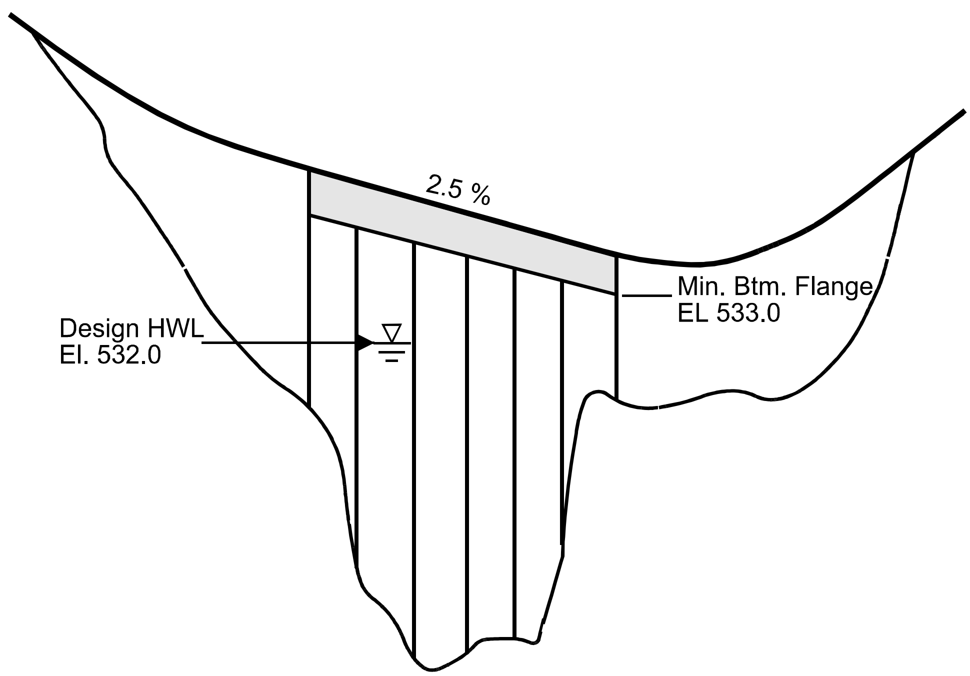
Estimates of general scour and local scour near piers for the hydraulic design of a bridge.

Given

- Design criteria
  - Design flow, Q_{100} (100 year return period) = 4358 m^3/s
  - To meet the requirement of the *Ontario Highway Bridge Design Code* (1991), the bridge foundation must withstand scour resulting from the check flood. The check flood flow is taken as 1.3 times the design flood flow. Check flow, $Q_c = 1.3 * Q_{100}$ = 5665 m^3/s
- River channel
  - A trapezoidal cross section with side slopes of 2:1. The bottom width is approximately 266 m at elevation 124 m.
  - Generally straight in the vicinity of the bridge site
  - Longitudinal path of the channel is fairly stable.
  - Mean channel invert slope = 0.0002
  - Manning's roughness coefficient, n = 0.024 (Design Chart 2.01)
- Hydraulic Conditions: Check flood
  - Typical river bed elevation = 124.0 m
  - Check flood high water level = 132.5 m
  - Average flow depth, d = 8.50 m
  - Waterway area at El.132.5 m = 2411 m^2
  - Mean velocity = 2.35 m/s
  - Effective top width of channel at El. 132.5 m = 300 m
- Bank-full conditions
  - Top-of-bank elevation = 130.0 m
  - Discharge, Q_b = 3100 m^3/s
  - Top width, W_b = 290 m
  - Depth of bank-full flow, d_b = 6.0 m

- Soil data:
  - 0 to 10 m below river bed, alternating layers of:
    - Gravel:
      - Particle size, D_{50}, = 3 mm
      - V_s depends on depth, approximately 1.6 m/s. (Design Chart 2.18)
    - Silty-sand:
      - Particle size, D_{50}, < 0.3 mm, D_{max} = 1.0 mm
      - assume cohesionless (conservative)
      - V_s = 1.0 m/s, assumed material to be highly scourable(Design Chart 2.18)
  - Clay shale, firm to hard at depths greater than 10 m below river bed
- Proposed Structure: see figure below
  - Six-span bridge with four interior spans of 60 m and two end spans of 45 m
  - Five piers to be constructed in the waterway
  - Abutments to be perched, well clear of the design high water level
  - Pier footings to be founded on sound foundation
  - Pier width 1.2 m each, aligned to flow, ends rounded, skew = zero

Figure: Profile of Crossing Site



Analysis: General Scour

Competent Velocity Method

The total depth of scour is:

$d_t = \quad d + d_{sa}$ (5.8)

where:

d_t = total scoured depth, measured from water surface, m
d = depth of flow before scour = 8.5 m (given)
d_{sa} = depth of scour, m

Assume that the lighter silty-sand material will be scoured out, leaving the heavier gravel as an armour layer. Therefore V_c = 1.6 m/s.

$A = Q_c / V_c$ = 5665 m^3/s / 1.6 m/s = 3541 m^2

Deepen the 300 m wide cross section such that the area is 3541 m^2. Scoured cross section area, A_s = 3541 m^2 - 2411 m^2 = 1130 m^2

Try three possible shapes for the scoured area:

(1) Trapezoidal Shape

Assuming a 266 m top width of the trapezoid and 2:1 slopes extending to the level bottom of the scoured area; d_s = 4.4 m.

(2) Triangular Shape, assuming a 266 m top width

d_{sa} = (1130 m^2 / 266 m) * 2 = 8.5 m

(3) Parabolic Shape, assuming a 266 m top width

d_{sa} = (1130 m^2 / 266 m) * 1.5 = 6.4 m

The layer of firm to hard clay shale will be encountered at depths of approximately 10 m below the river bed. General scour is not likely to progress into this layer. Since the channel is fairly straight longitudinally, use case 3, with a scour depth of 6.4 m. Therefore the general scour below the high water line is in the order of 14.9 m (8.5 m flow depth + 6.4 m scour depth), or elevation 117.6 m.

Mean Velocity Method

Bank-full average flow velocity, V_b

$V_b = Q_b / A_b = 3100\ m^3/s / 1670\ m^2 = 1.86\ m/s$

This method assumes that the bank-full flow condition is inherently stable. Therefore, the flow velocity during the bank-full flow condition represents the overall competent velocity for the channel lining material.

Therefore for the check flood flow condition, calculate the channel cross section area required to lower the flow velocity, V to V_b = 1.86 m/s.

$A = 5665\ m/s / 1.86\ m/s = 3046\ m^2$

Under the check flood flow condition, the cross-section would therefore scour out from the original 2411 m^2 until it covered an area of 3046 m^2, an increase of 635 m^2. The depth of the scoured area may be calculated similarly to the Constant Velocity Method discussed above.

(1) Trapezoidal Shape, assuming a top width of the scoured area of 266 m and 2H:1V sideslopes.

Scoured area, A_s = 3046 - 2411 = 635 m^2
Scoured depth = 2.43 m

(2) Triangular Shape, assuming a 266 m top width

Scoured depth = 2 * 635 m^2 / 266 m = 4.77 m

(3) Parabolic Shape, assuming a 266 m top width

Scoured depth = 1.5 * 635 m^2 / 266 m = 3.6 m

A scour depth of 3.6 m below the existing stream bed represents the parabolic cross section shape case. This depth of scour is equivalent to elevation (124.0 m - 3.6 m =) 120.4 m.

Regime Method

The following equation forms the basis for this method:

$$d_f = d_b * (q_c/q_b)^m \quad (5.9)$$

where:

d_f = total scoured depth (from bank-full water level), m

d_b = bank-full depth of flow before scour, m
q_c, q_b = flow intensities, m^2/s (m^3/s per m of width)
m = empirical exponent, depending on bed material
= 0.67 for sand, 0.85 for gravel

Calculate the discharge intensity, q, for both the check flow, Q_c, and bank-full flow, Q_b

$$q_c = Q_c / W_c = 5665\ m^3/s / 300\ m = 18.9\ m^2/s$$

$$q_b = Q_b / W_b = 3100\ m^3/s / 290\ m = 10.6\ m^2/s$$

Assume: m = 0.67 for sand

$$d_f = d_b\ (q_c/q_b)^m = 6.0\ (18.9 / 10.6)^{0.67} = 8.8\ m \qquad (5.9)$$

The total estimated scour depth is 8.8 m below top-of-bank, or elevation (130 m - 8.8 m =) 121.2 m. This is 2.8 m below the original typical river bed elevation of 124.0 m.

Laursen Method

This method may be applied where a constriction in a channel occurs, such as that caused by the approach fills of a bridge crossing. The following equation forms the basis of this analysis, assuming that the Manning's n values are the same for both cross sections:

$$d_2/d_1 = (Q_2/Q_1)^{6/7} * (w_1/w_2)^{0.64} \qquad (5.10)$$

where:

d_1 = average depth in the main channel = 8.0 m
d_2 = average depth in the contracted section, subject to scour, m
Q_1 = flow in the approach channel that is transporting sediment, m^3/s
Q_2 = flow in the contracted channel = 5665 m^3/s
w_1 = bottom width of the main channel = 266 m
w_2 = bottom width of the bridge opening, m

Analysis shows that, at this stage, approximately 1200 m^3/s flows in the over-bank area. Therefore, the main approach channel flow, Q_1, is approximately (5665 m^3/s - 1200 m^3/s =) 4465 m^3/s. The entire flow of 5665 m^3/s passes through the bridge waterway opening.

The five piers of the proposed bridge each have a width of 1.2 m, totalling 6.0 m. Therefore, the net bottom width, w_2, of the main channel at the bridge opening is reduced by 6 m with the proposed bridge piers in place to 260 m.

Solve for d_2, using equation 5.10 and the above input:

$$d_2/d_1 = (Q_2/Q_1)^{6/7} * (w_1/w_2)^{0.64} \qquad (5.10)$$

Rearranging:

$$\begin{aligned} d_2 &= (Q_2/Q_1)^{6/7} * (w_1/w_2)^{0.64} * d_1 \\ d_2 &= (5665/4465)^{6/7} * (266/260)^{0.64} * 8.0 \text{ m} \\ &= 1.226 * 1.015 * 8.0 \text{ m} \\ &= 9.95 \text{ m} \end{aligned}$$

Calculate Depth of Scour, d_{sa}

$$\begin{aligned} d_{sa} &= d_2 - d_1 \qquad (5.11) \\ &= 9.95 \text{ m} - 8.0 \text{ m} \\ &= 1.95 \text{ m} \end{aligned}$$

This assumes an equal depth of scour across the cross section.

If the scoured cross section is parabolic shaped, the maximum depth of scour will be (1.95 m * 1.5 =) 2.9 m, while for a triangular distribution, it will be (1.95 * 2 =) 3.9 m. A depth of scour of 2.9 m, will provide a general scour elevation of (124.0 - 2.9 =) 121.1 m.

Method	General Scour Elevation, m
Competent Velocity Method	117.6
Mean Velocity Method	120.4
Regime Method	121.2
Laursen Method	121.1

As indicated above, the methods used for estimating general scour yield varying results. The Competent Velocity Method provides the most conservative result with 6.4 m of scour depth, while the Regime Method provides 2.8 m of scour. The discretion of the engineer is paramount in evaluating the use of any analytical method for a particular design situation. The average depth of the four methods is 3.9 m below the river bed, to an elevation of 120.1 m.

Analysis: Local Scour

Melville and Sutherland (1988) Method

This method is applied to estimate the maximum local scour to be expected in the vicinity of a pier of the proposed bridge.

The following equation was used to determine the depth of local scour below the bed, d_{sp}.

$$d_{sp} / b_p = k_i * k_y * k_d * k_\sigma * k_s * k_\alpha \qquad (5.7)$$

where:

b_p = width of pier normal to flow = 1.2 m (given)
k_i = flow intensity coefficient
k_y = flow depth coefficient
k_d = sediment size coefficient
k_σ = sediment gradation coefficient
k_s = pier shape coefficient
k_α = pier alignment coefficient

The average flow depth with the check flood, y = 8.0 m. The corresponding mean flow velocity is 2.35 m/s. The median grain size of the silty-sand bed material, D_{50} = 0.3 mm and the maximum grain size, D_{max} = 1.0 mm.

1. Determine σ (sigma):

$$\sigma = D_{max} / D_{50} \qquad (5.12)$$

where:

D_{max} = 1.0 mm
D_{50} = 0.3 mm

$$\sigma = 1.0 \text{ mm} / 0.3 \text{ mm} = 3.3$$

2. Estimate D_{50a}:

$$D_{50a} = D_{max} / 1.8 = 1.0 \text{ mm} / 1.8 = 0.56 \text{ mm} \qquad (5.13)$$

3. Determine shear velocity, u_{*c} and u_{*ca} (Design Chart 5.10)

u_{*c} = 0.03 m/s, where D_{50} = 0.3 mm

u_{*ca} = 0.07 m/s, where D_{50a} = 0.56 mm

4. Determine U_c and U_{ca}:

$$U_c / u_{*c} = 5.53 * \log(y / D_{50}), \quad (5.14)$$

where: flow depth, y = 8.0 m (8000 mm)

Rearranging:

$$\begin{aligned} U_c &= 5.53 * \log(y / D_{50}) * u_{*c} \\ &= 5.53 * \log(8000 \text{ mm} / 0.3 \text{ mm}) * 0.03 \text{ m/s} \\ &= 0.74 \text{ m/s} \end{aligned}$$

Similarly,

$$U_{ca} / u_{*ca} = 5.53 * \log(y / D_{50a})$$

where: flow depth, y = 8.0 m (8000 mm)

Rearranging:

$$\begin{aligned} U_{ca} &= 5.53 * \log(y / D_{50a}) * u_{*ca} \\ &= 5.53 * \log(8000 \text{ mm} / 0.56 \text{ mm}) * 0.07 \text{ m/s} \\ &= 1.6 \text{ m/s} \end{aligned}$$

5. Estimate $U_{a:}$

$$\begin{aligned} U_a &= 0.8 * U_{ca} \\ &= 0.8 * 1.63 \text{ m/s} \\ &= 1.3 \text{ m/s} \end{aligned}$$

Verify relationship: $U_a > U_c$
U_a = 1.3 m/s > 0.74 m/s = U_c

6. Estimate $\underline{U}$:

$$\begin{aligned} \underline{U} &= \{U - (U_a - U_c)\} / U_c \\ &= \{2.35 - (1.3 - 0.74)\} / 0.74 \\ &= 3.4 \end{aligned}$$

where: mean flow velocity, U = 2.35 m/s

If $\underline{U}$ is greater than 1.0, use k_i = 2.4

7. Estimate the sediment size factor, k_d, (Design Chart 5.12)

   where: pier width, b_p = 1.2 m and D_{50} = 0.3 mm.
   $k_d = 1.0$

8. Estimate the flow depth factor, k_y, (Design Chart 5.11)

   where: y = 8.0 m and b_p = 1.2 m
   $k_y = 1.0$

9. Estimate the shape factor, k_s, (Design Chart 5.13)

   where: pier shape (plan view) is rectangular with length / width ratio = 6.0
   $k_s = 1.1$

10. Sediment size factor, k_σ,
    recommended as 1.0

    $k_\sigma = 1.0$

11. Determine the pier alignment factor, k_α, (Design Chart 5.14)

    where: the pier angle of attack relative to the mean flow direction is near zero, $\alpha = 0^o$.
    $k_\alpha = 1.0$

12. Substitute the above values in the following equation and solve for d_{sp}:

    $$d_{sp} / b_p = k_i * k_y * k_d * k_\sigma * k_s * k_\alpha \quad (5.7)$$

    where:

    $b_p = 1.2$ m
    $k_i = 2.4$
    $k_y = 1.0$
    $k_d = 1.0$
    $k_\sigma = 1.0$
    $k_s = 1.1$
    $k_\alpha = 1.0$

    Rearranging:

    $$\begin{aligned} d_{sp} &= k_i * k_y * k_d * k_\sigma * k_s * k_a * b_p \quad (5.7) \\ &= 2.4 * 1.0 * 1.0 * 1.0 * 1.1 * 1.0 * 1.2 \text{ m} \\ &= 3.2 \text{ m} \end{aligned}$$

The estimated depth of local scour in the vicinity of a pier is 3.2 m.

RTAC Guide to Bridge Hydraulics Method

This method calculates local scour depths to be encountered adjacent to bridge piers. The local depth of scour is in addition to any general scour resulting from a waterway constriction.

Given: Pier width, w = 1.2 m (rounded pier ends)
Cohesionless soil

Therefore:

Local Scour depth, d_s = 1.5 * w (Design Chart 5.08)
= 1.5 * 1.2 m
= 1.8 m

The local scour depth is in addition to any predicted general scour depth.

Colorado State University Method

The following equation forms the basis for this method:

$$y_{ls} = 2.0 * k_1 * k_2 * (a / y_1)^{0.65} * F_r^{0.43} \quad (5.6)$$

where:

y_{ls} = depth of local scour, m
Pier shape correction factor, k_1 = 1.0 (for: round nose) (Design Chart 5.09)
Angle of attack correction factor, k_2 = 1.0 (for: angle of attack = 0) (Design Chart 5.09)
Width of pier, a = 1.2 m
Upstream flow depth, y_1 = 8.5 m
Froude number, F_r = 0.26 (with V = 2.35 m/s)

$$y_{ls} = 2.0 * 1.0 * 1.0 * (1.2 / 8.5)^{0.65} * 0.26^{0.43}$$
$$= 0.31 \text{ m}$$

The local depth of scour is 0.31 m adjacent to a pier. This depth is in addition to any predicted natural scour.

Table: Comparison of Results

Local Scour Method	Additional Scoured Depth (m)
Melville and Sutherland Method	3.2
RTAC Method	1.8
Colorado State University Method	0.31

The above methods of estimating scour also show wide variance in results. The average of the results from the RTAC and Melville & Sutherland Methods provides a local scour depth of 2.5 m.

Design Scour Levels

Based on the above estimates, the following conclusions are reached:

For abutments and rock protection:
Design Scour Elevation = 120.1 m.

For pier design:
Design Scour Elevation = 117.6 m.

Fish Passage in Culverts

Culvert crossings may become a barrier to fish passage, mainly due to increased velocities and the lack of resting areas for travelling fish. This is especially the case for a culvert with a closed invert. The best remedy is to avoid creating a barrier. If this is not practical, a fish passage should be provided in the culvert.

In some cases, a fish passage may be created by suitably adjusting certain hydraulic properties of the culvert and no baffles are required. This could be achieved by embedding the invert and/or widening the waterway area to increase the cross-section area, thereby reducing flow velocities. Increased surface roughness (Manning's n) will reduce flow velocities and enhance fish passage. In other cases, fish baffles are needed. Regardless of the culvert configuration, fish must be able to pass during periods of low flow. For this purpose, minimum depths of flow are specified.

Fish Baffles

Baffles are hydraulic devices (boulders or formed blocks / shapes) placed at intervals in a culvert along its length to dissipate energy and reduce flow velocities. Also, the areas behind baffles provide resting areas for migrant fish. Figures 5.12 - 5.17 show fish baffles in-place in culverts. The usual spacing between such baffles along the barrel is two to three times the culvert diameter. These are often used where invert slopes are relatively steep (0.5 to 5.0%), or where a culvert may otherwise create unusually high flow velocities. Careful placement of large boulders along a barrel invert may produce desirable results.

The following types of baffles are generally used for fish passage

- For concrete culverts: slotted weir, weir baffle and offset baffle; refer to Figures 5.12 and 5.13. For steel culverts: slotted weir, weir baffle, offset baffle, spoiler baffle and fish weir; refer to Figures 5.14 to 5.16.

With contribution by Bender (1996)

Figure 5.12: Slotted Weir Baffles in a SPCSP Culvert

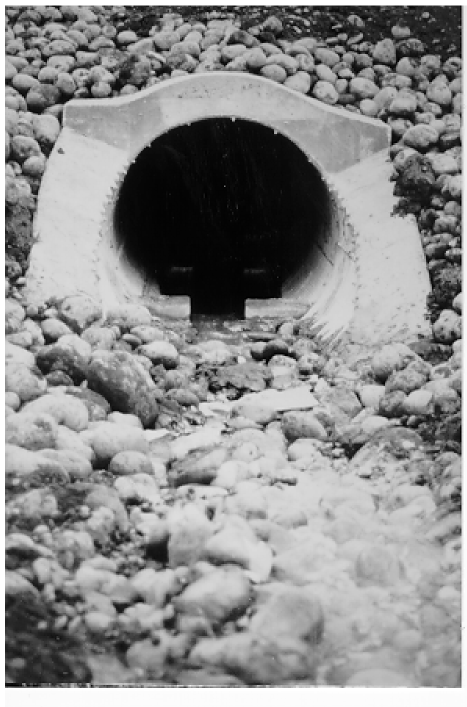


Figure 5.13: Weir Baffles in Concrete Culvert

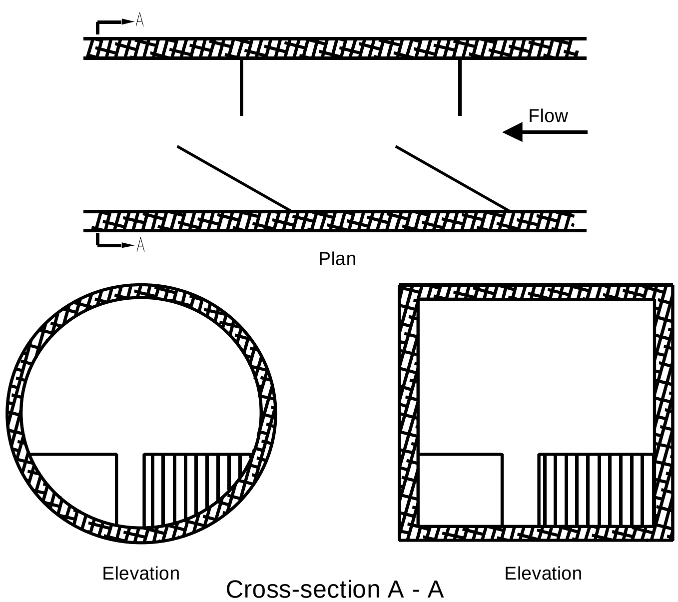


Figure 5.14: Slotted Weir Baffles in a Steel Culvert

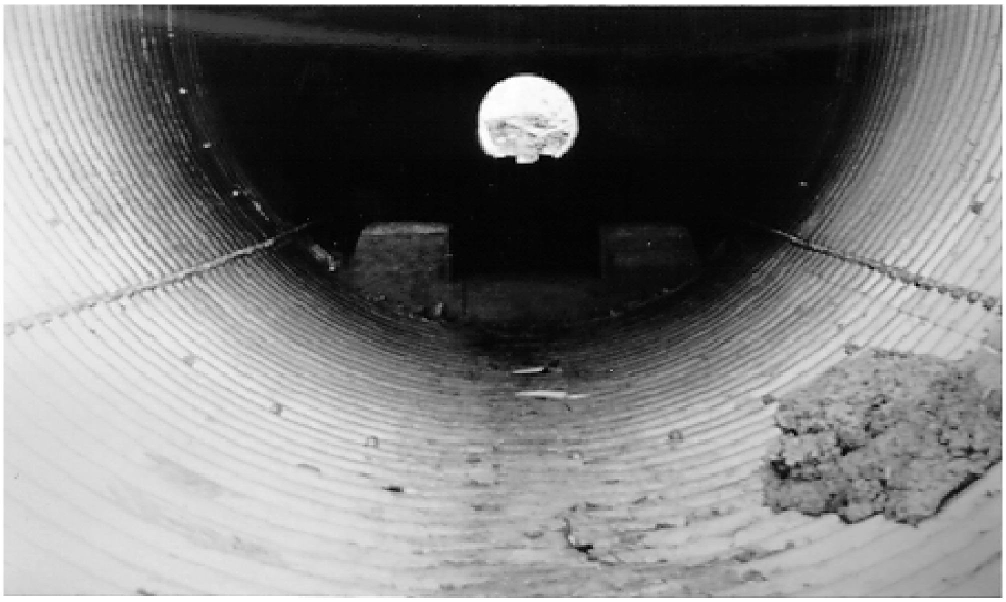


Figure 5.15: Spoiler Baffle in a Steel Culvert

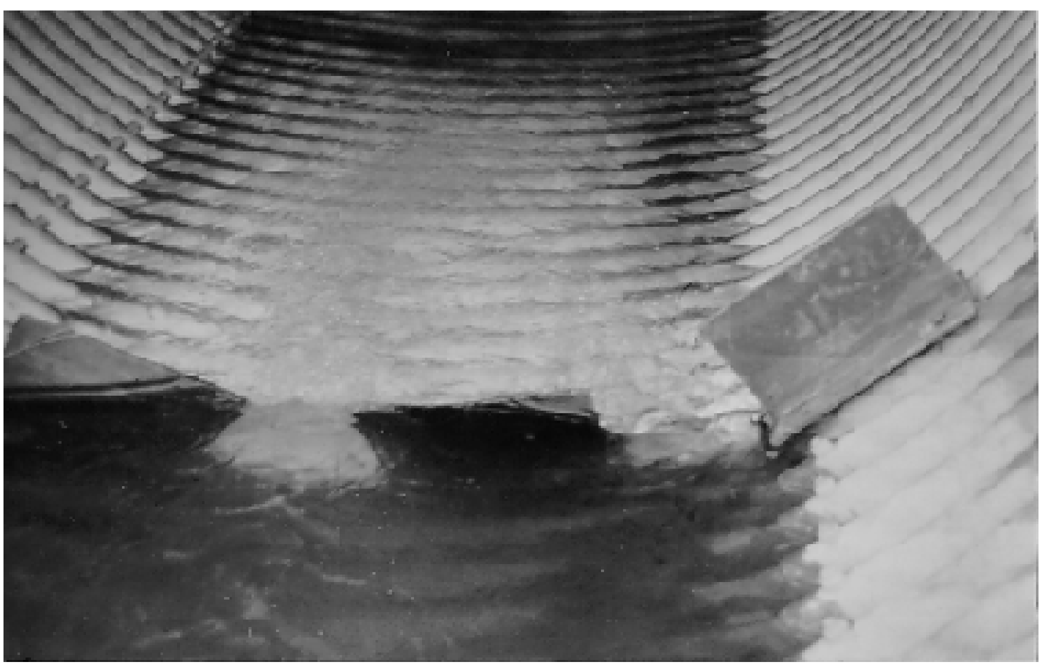


Figure 5.16: Weir Baffles in a Steel Culvert

Hydraulic Design Considerations

- A designer should work closely with a fish biologist to ensure that the hydraulic design meets the biological requirements of the fish.
- As a starting point, note this general information about fish migration behaviour.
    - Migrating fish are generally attracted by flow patterns. They use the patterns to identify travel and migration paths. Excessive ponding may "hide" a culvert from migrants.
    - Generally, fish have limited swimming ability, which varies with species, maturity, and motivation.
    - Fresh water species in Ontario generally are not strong leapers and perform poorly at obstacles requiring large jumps. Special provision may be needed for both juvenile and adult fish. Juvenile fish are generally weaker swimmers than adult fish.
- For streams prone to ice and debris problems, culverts should be avoided.
- A fish passage in a culvert may be created in two ways.
    - Modify an existing or a pre-selected culvert section. This may be done by increasing the size of opening, length or roughness.
    - Provide baffles to create desired areas of current and resting areas.

- For a design using multiple culverts, at least one culvert may be used for fish passage. The culvert inverts may be placed at different elevations. The invert of the culvert providing fish passage should ideally be set below the stream invert.

- Make an effort to design a culvert to reasonably mimic the existing stream characteristics in terms of cross-section, slope and roughness. This may be attained by either;

  - installing an open footing culvert, leaving the stream section generally undisturbed, or
  - Alternatively, a culvert may be embedded below the stream invert and the embedded cross-section area filled to the stream bed elevation with a non-uniform substrate material. A fish biologist may be able to provide advice.

- A culvert should be designed to convey fish passage flows under outlet control conditions. Inlet control should be avoided as it may result in excessive ponding upstream of a culvert, unstable flow, high velocities at the inlet and scour at the outlet. These conditions are unfavourable for fish passage.
- The design discharge, flow depth and velocity for fish passage should not exceed the limiting biological requirements of the fish. Depths that are too shallow may cause delay of migration or injury to the fish. A combination of shallow depth and low velocity may result in reduced mixing of the flow and higher water temperatures which are also unfavourable to fish.
- The following is a general guide for determining limiting values of design parameters.

  - Maximum head loss for fish passage flow = 0.3 m.
  - Minimum flow depth = 0.2 m.
  - Minimum embedment of invert = 0.3 m.
  - Maximum culvert length = 60 m.
  - Minimum resting pool depth = 0.3 m.
  - Maximum invert slope (Preferably to match existing stream invert) for:
    - unmodified culverts = 0.5%; or
    - baffled culverts = 5.0%
  - Culvert barrel sizes less than 3.0 m are less desirable.

- Where flow velocities exceed the fish passage velocity, fish baffles may be required to reduce velocities. The effect of fish baffles on culvert flows may exceed that of simply blocking the corresponding part of the culvert cross-section. Provision of fish baffles may cause a culvert to flow under outlet control that would have otherwise operated under inlet control.
- For ponded or slow flowing areas, the flow velocity may be increased with a combination of barrel slope and baffles.
- For sensitive fish, open footing culverts are desirable.
- Channel stabilization measures at culvert ends and invert embedment should be designed to ensure that excessive scour pools obstructing fish movement will not form.
- The culvert outlet should not be excessively high or fish may have trouble entering from ownstream. However, fish need to detect flow (velocity) to guide them in their upstream

travel. High outlet velocities may lead to excessive erosion and destruction of fish habitat.

- Excessive backwater upstream may result in loss of spawning areas due to sedimentation and stagnant flow. Also, fish may become stranded if flows and water levels decrease.

Fish Passage Computations

In the design of all fish passages, it is necessary to determine the fish passage design flow. The migratory season and permissible time delay (the time interval for which the fish may be able to tolerate short delays in migration) must be known for the species of interest. This information is usually obtained from a fish biologist. A method for determining a fish passage design flow is available in Chapter 8.

The hydraulic method for fish passage design depends on whether a fish baffles are needed or not. The MTO Open Channel Model computer program may be a useful computation tool.

Fish Passage Not Requiring Baffles

1. For a trial culvert waterway opening, estimate flow depth, d_p, and velocity, V_p, for the fish passage flow, Q_p, using the Manning equation (8.66). To start, assume the culvert invert is matched to the existing channel invert.

   If fish passage criteria are not satisfied, the culvert invert may be lowered and embedded into the stream invert. Add the proposed culvert embedment depth to the calculated normal flow depth to obtain a revised depth of flow, d_p. Recalculate the fish passage flow velocity, V_p as Q_p/A where A is the revised wetted cross section area.

   For a round culvert with a depressed invert under a partially full flow condition (Jordan and Carlson, 1987), estimate.

   $$F = 2\cos^{-1}(1 - 2D_d/D) \quad (5.15)$$
   $$R = 2\cos^{-1}[1 - 2(d + D_d)/D] \quad (5.16)$$

   where:

   D_d = Depressed depth below invert, m
   D = Culvert diameter, m
   d = depth of flow to stream invert, m
   F = perimeter of culvert below stream invert, radians
   R = perimeter of culvert below water level, radians

   The cross section area of the culvert above the stream bed and below the water level, A_b, is:

   $$A_b = (D^2/8) * [R - \sin R - (F - \sin F)] \quad (5.17)$$

The wetted perimeters for the culvert bed, p_b, and culvert walls, p_w, are:

$$p_b = D \sin (F/2) \tag{5.18}$$
$$p_w = D/2 (R - F) \tag{5.19}$$

The hydraulic radius, r_d, of a culvert with a depressed invert is:

$$r_d = A_b / (p_b + p_w) \tag{5.20}$$

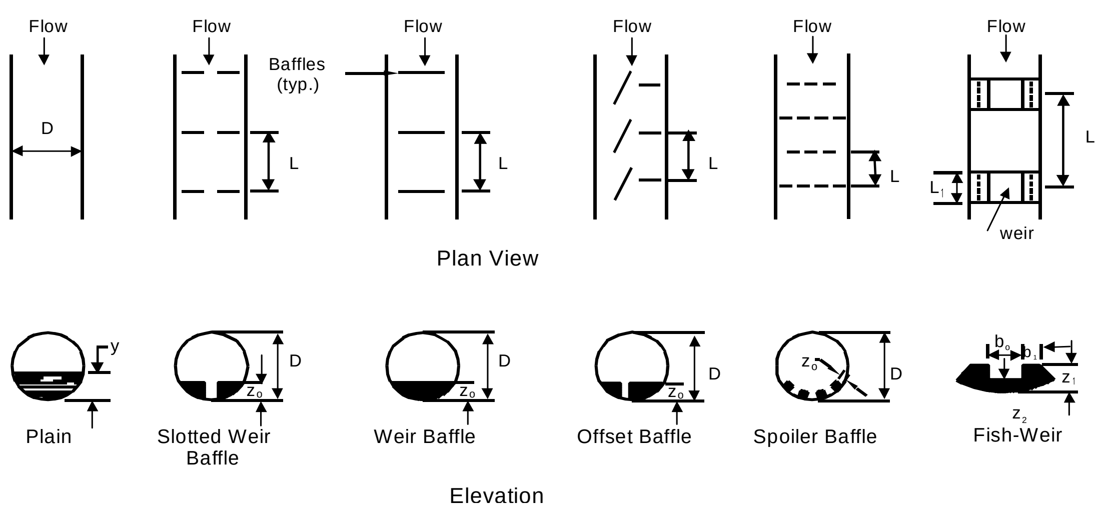
2. Check whether the computed flow depth and velocity satisfy design criteria. If not, revise the design dimensions and repeat Step 1.

Fish Passage Requiring Baffles

If the requirements for fish passage are not met by following the above steps, such that a reasonable combination of depressed culvert barrel and increased cross-section area does not provide the required depth of flow and flow velocity, specially designed baffles may be required (See Figure 5.17, below). Other types of baffles (not shown) include those formed using boulders, aligned in rows or clusters. Only smooth, well rounded rock should be used so that potential injury to fish is reduced.

A computation procedure for hydraulic design of a fish passage with baffles is shown in Example 5.7.

Figure 5.17: Baffle Height and Spacing



Design Example 5.7
Fish Passage in Culverts

Required

Design a fish passage for a culvert crossing. Consider two options; a rectangular concrete box culvert and a circular SPCSP culvert.

Given

- Migratory Fish Characteristics:
    - Fish are able to negotiate a 35 m length upstream against a flow of 0.8 m/s
    - Permissible time delay in migration due to excessive flow velocity, T = 3 days
    - Migration season: May 1 to June 30

- Common Culvert Parameters
    - Length, L = 35 m
    - Longitudinal culvert slope, S = 0.005
    - Culvert outlet not submerged during low flow.

- Box Culvert Parameters
    - Trial culvert opening: width, w = 6.0 m; height, d = 4.0 m; may be revised depending on fish passage requirements.
    - Manning's n = 0.013 (Design Chart 2.01)
    - Culvert invert embedment below stream invert, d_e = 0.3 m

- Circular SPCSP Culvert Parameters
    - Trial culvert diameter, D = 6.0 m
    - Culvert invert embedment below stream invert, d_e = 0.6 m
    - Manning's n = 0.023 (Design Chart 2.01)

Analysis

Fish Passage Flow

For discussion on methods for estimating fish passage flow, see Chapter 8.

Examine the daily mean flow of records for 1978 to 1992 (15 years) obtained for the stream flow gauging stations to determine the consecutive 3-day highest flow for May and June for each year of record. For example the 3-day flow for June, 1980 is 2.4 m^3/s as shown in the table below.

Table: Mean Daily Flows, June 1980, at the Subject Site

date	flow (m^3/s)	Date	flow (m^3/s)	date	flow (m^3/s)
1	1.5	11	1.5	21	1.7
2	1.9	12	1.4	22	1.4
3	1.7	13	1.2	23	1.5
4	1.3	14	0.9	24	1.6
5	**5.2**	15	0.7	25	4.7
6	**3.1**	16	0.7	26	2.9
7	**2.4**	17	2.2	27	2.2
8	1.9	18	1.7	28	1.9
9	1.5	19	1.7	29	1.8
10	1.3	20	1.4	30	1.5

The data show that the highest three (T) consecutive flows occur on the 5th, 6th and 7th days of the month. For this month, the T-day flow is 2.4 m^3/s, the lowest flow over the three consecutive days.

In a similar manner, determine the 3-day time delay discharge for the migratory season of May and June.

Rank the yearly 3-day time delay flows from largest to smallest with ranks 1 to 15. A statistical frequency analysis (3-parameter lognormal) was done for a 10-year return period The resulting fish passage flow is 2.9 m^3/s.

Statistical analysis is presented in Chapter 8.

Fish Passage: Box Culvert

From a hydraulic analysis for a flow rate of 2.9 m^3/s, the flow depth downstream of the culvert is 0.4 m. The results are listed below:

Inlet depth of flow, d = 0.63 m
Maximum flow velocity, V = 1.2 m/s

The above estimates indicate that the velocity for fish passage flow exceeds the 0.8 m/s allowable for fish migration. Consider possible options to reduce the flow velocity and/or enhance fish passage:

- Lower the culvert invert to provide a greater depth of flow and hence a greater flow cross section with a corresponding reduced flow velocity.

- Increase culvert width to provide greater conveyance and reduce flow velocities.
- Increase hydraulic roughness of the culvert by installing baffles at specified intervals. This will reduce flow velocity and increase flow depth. Fish baffles would provide rest areas for migrating fish.

Try increasing the depth of embedment, d_e to 0.6 m. The hydraulic analysis shows the following results:

Inlet depth of flow, d = 0.76 m
Maximum flow velocity, V = 0.67 m/s

This hydraulic condition is acceptable as it meets the fish passage requirement for flow depth and velocity.

Fish Passage: Circular SPCSP Culvert

Try a circular SPCSP culvert with an embedment, d_e of 0.6 m below the natural stream invert. The analysis results are:

Inlet depth of flow, d = 0.95 m
Maximum flow velocity, V = 1.41 m/s

The above flow velocity estimates exceed fish passage requirements.

Try a culvert embedment, d_e of 0.9 m. The analysis results are:

Inlet depth of flow, d = 1.11 m
Maximum flow velocity, V = 0.90 m/s

This trial shows that the culvert flow velocity still exceeds the maximum allowable fish passage flow velocity of 0.8 m/s. Try again with fish baffles.

Estimate fish baffle parameters:

$$I = Q_p / (g * S_o * D^5)^{1/2}$$

where:

Fish Passage T-day Flow, Q_p = 2.9 m^3/s
Acceleration due to gravity, g = 9.81 m/s^2
Longitudinal Slope, S_o = 0.005
Culvert Diameter, D = 6.0 m

$I = 2.9 / (9.81 * 0.005 * 6.0^5)^{1/2}$
$= 0.148$

$J = 0.28$ (Design Chart 5.18)

where:
$I = 0.148$, assuming 'slotted weir' type baffles

$d_p = J * D$
$= 0.28 * 6.0$
$= 1.68$ m

$K = 0.24$ (Design Chart 5.19)

where:
$J = 0.28$, assuming 'slotted weir' type baffles

$U_m = K (g * S_o * D)^{1/2}$

where:
$K = 0.24$
Acceleration due to gravity, g $= 9.81$ m/s^2
Longitudinal Slope, S_o $= 0.005$
Culvert Diameter, D $= 6.0$ m

$U_m = 0.24 (9.81 * 0.005 * 6.0)^{1/2}$
$= 0.13$ m/s

Compare d_p versus d:
$d_p = 1.68 \text{ m} > 1.11 \text{ m}$ = allowable depth of flow, d

Compare U_m versus V:
$U_m = 0.13 \text{ m/s} < 0.90 \text{ m/s}$ = allowable velocity, V

The parameters d_p and U_m are both within acceptable limits as shown above. Therefore the provision of fish baffles in this circular SPCSP will allow fish passage as required. Slotted weir type fish baffles are recommended for this application because they will pass low flows and will not trap water during periods of low flow. Additionally, they will allow the passage of smaller debris.

The longitudinal spacing of the baffles is usually 2 to 3 times the culvert diameter. Therefore, with a culvert diameter of 6.0 m, a spacing of 12 m to 18 m is appropriate. Three baffles installed in the 35 m long culvert will be sufficient, one at the midpoint and one at a distance of 5 m from each end. This will provide a 12.5 m interval between baffles.

The baffles should be at least 0.3 m submerged for the fish passage flow Q_p of 2.9 m^3/s. With a flow
depth of 1.11 m, the baffles should be a maximum of 0.81 m high. A height of baffle of 0.75 m above the culvert invert is specified. The width of the slot in each baffle is 0.6 m.

Recheck the flow characteristics of the culvert. For the design flow of 30 m^3/s, and a corresponding tailwater level of 4.3 m above the downstream invert, the hydraulic analysis shows the following results:

Table: Design Hydraulic Conditions

	*HW (m)	Mean V (m/s)
Without Baffles	4.28	1.39
With Baffles	4.34	1.37

* depth measured from upstream invert

Analysis, using the CULVFLOW computer model, shows that the upstream water level for the design discharge condition will increase by 0.06 m and culvert mean flow velocities are slightly reduced due to the presence of the baffles. This may be conservatively be handled by blocking out the baffle cross section area. However, additional analysis should be undertaken if the culvert would flow in inlet control without baffles and the presence of the baffles causes the culvert to flow in outlet control.

River Ice

In Ontario, streams with continuous flows year round have ice cover with water continuing to flow under the ice (Figure 5.18) Intermittent water courses may freeze to the channel bed with thin layers of water flowing over and freezing.

Ice problems usually occur during spring melt and runoff. Problems are associated with ice flow, ice jamming and channel icing. In designing water crossings and stream channels, designers should assess potential ice problems and work out solutions where necessary.

Freeze-up and Break-up Processes

Stream freeze-up produces a mass of ice on a river. Break-up of river ice may result in ice jams and ice forces on water crossings, embankments and stream channels. Rivers and streams with higher flow velocity and mixing, tend to freeze at a slower rate.

Figure: 5.18: Frozen ice Cover in a Stream Channel

Flocks of ice or frazil ice tend to form and join together in clusters. With further cooling, the top ice layer may grow in thickness and strength. As large flocks and clusters form, they may anchor to the channel bed in supercooled flowing water.

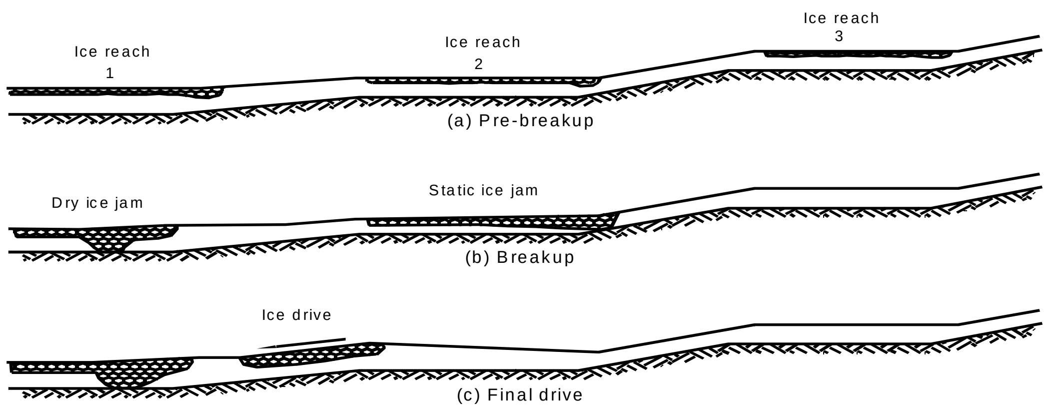
For a flowing stream, the water's edge may freeze earlier than the deeper middle portions. In wide water bodies, such as lakes, freeze-up usually proceeds from the shallower edges toward the inner, deeper portions. Large water bodies may not be fully ice covered. Figure 5.19 illustrates a typical process of ice formation in lakes and rivers.

Figure 5.19: Freeze-up and Break-up Processes

Break-up of ice cover occurs when warmer weather occurs in the spring. At the same time, due to snowmelt and rainfall in the upstream watershed, channel flows increase. This usually results in an increase in flow depth below the ice cover, lifting it up and breaking it at its edges. With a further increase in flow depth, ice may be lifted higher and may break up in large sheets and start to flow downstream. Depending on weather patterns over a watershed, ice break-up may proceed upstream or downstream. In the early stage of a break-up, ice may move in large sheets or may break into smaller sheets, as it crushes against channel banks and obstructions. The features of break-up on a river reach are shown in Figure 5.20.

Severe ice jams often occur in rivers flowing north into James Bay or Hudson Bay because the upper reaches melt first and flow downstream to the still-frozen lower reaches where the longitudinal slope is nearly zero.

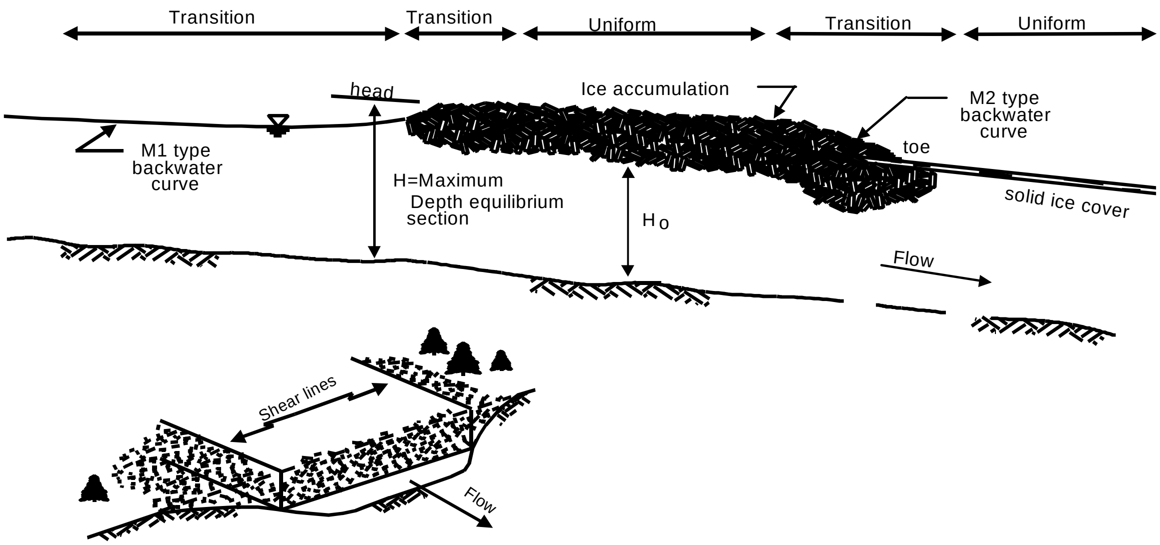
Figure 5.20: Ice Break-up in a River Reach



Ice jams may occur during both freeze-up and break-up conditions, however, the most severe ice jams occur at break-up. An ice jam may form under the following conditions:

- Floating pieces of ice may be held in place by an obstruction in a channel section. For the ice accumulation to be stable, it must resist all drag and other forces exerted by flows. The ability of a cluster of ice sheets to resist movement is dependent on the strength and thickness of ice sheets.
- As ice fragments continue to arrive from the upstream reach, the size and strength of the accumulated conglomeration of ice sheets also increases.
- With ice fragments continuously arriving from upstream, the size of the jam may grow. Upstream of the toe of a jam, the accumulation of ice may become rougher and thicker. A jam may be strengthened by freezing together of the surfaces of ice sheets. However, this freezing does not always occur.
- The height of the jam may increase until the forces acting on it exceed the resistance forces, resulting in breakage of the jam. The type of ice accumulation profile that results in a jam is illustrated in Figure 5.21.

Figure 5.21: Typical Ice Jam Profile



Ice-Related Problems

Ice related problems in stream channels such as ice abrasion, jamming and flooding are generally caused by ice break-up, rather than by freeze-up.

Ice Jams

An ice jam may occur when ice moves downstream in the following conditions:

- constriction of the flow, such as a typical highway crossing,
- obstruction of the flow, such as an island or bar,
- channel bend where the radius of curvature, R_b, is less than four times the channel bed width, w.
- an upper reach is experiencing ice breakup while the downstream reach is still solidly frozen. In that case, the edge of the solid ice sheet downstream, may initiate a jam by obstructing the flow of ice sheets from upstream. This condition is quite prevalent at the confluence of a stream with a larger river.

Ice jams are caused by the flowing ice sheets arrested due to arching between the banks. As a result, the ice may tend to accumulate to a substantial thickness below the water surface and, in some cases, up to the channel bed. The resulting ice jams may cause the following types of problems:

- Increased scour at waterway constrictions or relief flow paths,
- Flooding upstream of an ice jam and aggravated channel scour downstream resulting in damage

to lands and properties,
- Water levels higher than free flow design HWLs,
- Damage to structures due to ice abrasion,
- Impact of ice forces on bridges, abutments and piers which could result in structural damage or destruction,
- Channel icing which may reduce the conveyance capacity of a water crossing, resulting in upstream flooding,
- Risk of public safety,
- Surges of flow from the sudden release of jams may aggravate these problems.

Ice Forces

Flowing ice may result in the following types of forces on structures:

- Horizontal forces acting on piers, abutments and embankments due to impacts of ice sheets. The forces may act longitudinally, transversely or obliquely. The forces are generally horizontal, however, a vertical component may be generated on sloping surfaces. These forces may be static, caused by contraction and expansion of ice sheets or dynamic due to impact caused by moving ice sheets.
- Vertical forces caused when water levels vary and ice sheets are frozen around piers.
- Lifting of superstructure from ice contact due to insufficient clearance.
- Ice impact may also result in undesirable vibrations in bridges.

Flowing Ice Conditions

Studies and experience have shown that forces due to flowing ice conditions may vary:

- with site conditions,
- flow rate and ice thickness,
- weather conditions prior to breakup; such as temperature, sunlight and precipitation.

According to *Canadian Standard Association Manual, CAN3-S6-M78, Ice Forces*, and the *Ontario Highway Bridge Design Code* (1991), moving ice conditions in a stream may be categorized as one of the following.

- **Situation "a"** Break-up occurs at melting temperatures and ice flows in small cakes and its structure is substantially disintegrated.
- **Situation "b"** Break-up occurs at melting temperatures, however, ice moves in large pieces and is internally sound.
- **Situation "c"** Break-up and ice movement occurs in a single sheet or large sheets of ice.
- **Situation "d"** Breakup occurs with ice temperature significantly below the melting point and

the movement of ice is predominantly in the form of single or very large ice sheets.

Assessing River Ice Conditions

Assessment of river ice conditions should include the following aspects:

- Severity of the flowing ice situation to determine whether a site under consideration is prone to significant ice problems and its suitability for a water crossing. For reaches subject to ice runs, locations to be avoided include;

    - the outside of a bend,
    - a split-channel with large bars immediately upstream of a confluence,
    - near a location historically known for ice jams.

- Maintaining a flow alignment which will not aggravate the flowing ice situation.
- The design high ice condition including ice level, ice thickness, and situations 'a' to 'd'.
- Selecting between a bridge versus culvert or, for larger waterways, a suitable bridge span arrangement and pier locations.
- Assessing the potential for ice jams and how they may be avoided. If a jam can't be avoided, what is the design condition which must be considered?
- Channel icing and potential for blockage and how this should be accommodated in design.
- Risks to a structure, stream channel or public safety and mitigative measures to alleviate such risks.

Assessing Flowing Ice Condition

Methods for assessing flowing ice conditions are outlined below.

Past High Ice and Flooding

The history of ice problems may sometimes be determined by observing ice scars on trees (Figure 5.22). Older trees along the banks will often bear ice scars covering a long time duration. Ice abrasion and impact marks may be found on stream banks, bridge piers, abutments and girders. Interviews of long-term residents to discuss their recollection of past ice runs may be very helpful. They may be able to provide good information on high water levels and ice thicknesses. Some may have photographs of extreme events. In urban situations, information may be found in the municipal records, libraries or archives.

If there was a recent ice run, evidence may be detected from debris pushed up by ice floes against the banks. The elevations of ice marks may be determined and this information may then be used to validate the design condition.

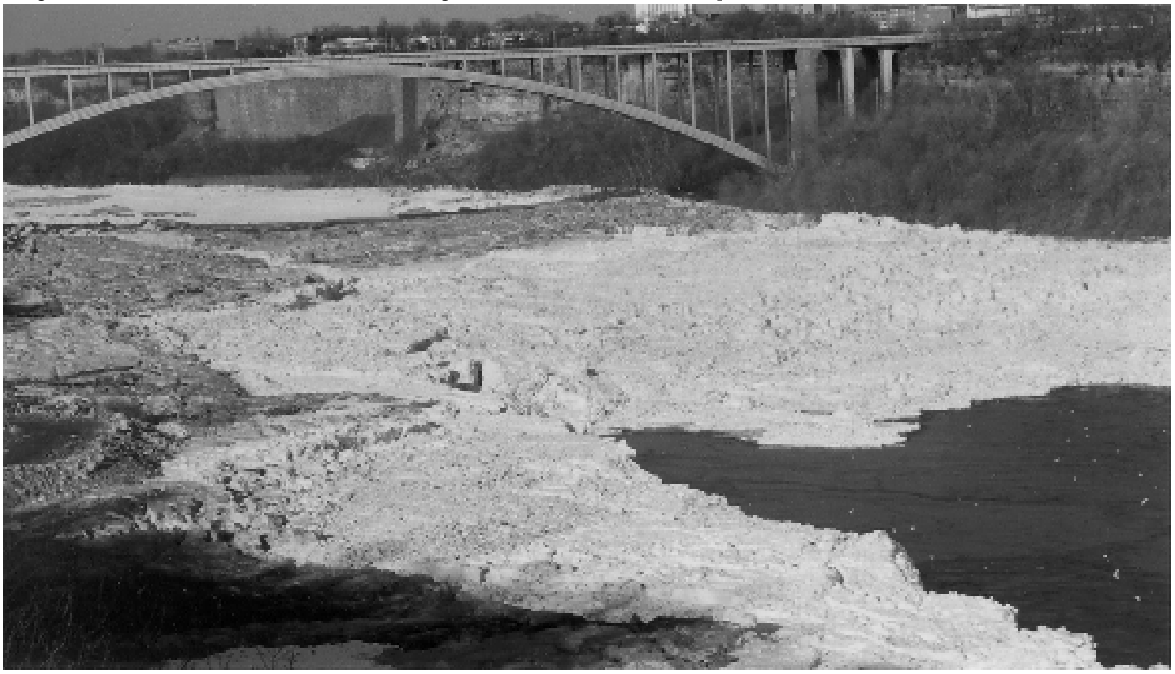
Figure 5.22: Ice Scar on a Tree

Ice Thickness

The thickness of flowing ice may be determined from historical information using the approach shown below (Figure 5.23). Additionally, Environment Canada maintains historical records of ice thickness measurements at selected streams and lakes. If such records are available for another river with similar characteristics and climatic setting, they may be used to determine the design conditions at the site under consideration.

Ice thickness may also be measured prior to breakup by boring through an ice layer. The measurement should account for additional thickness due to white ice (compacted snow at the top, frazil at the bottom end). This one-time observation, when compared with long-term data of a neighbouring stream, may aid in providing a correlation.

Figure 5.23: Ice Floes Resting on a River Floodplain



Estimating Design High Ice

Design high ice conditions are estimated to ensure that the waterway opening of a bridge crossing is large enough to pass flowing ice and to incorporate mitigative measures to handle adverse effects. Specifically, an estimate of design high ice may be used for:

- Establishing the minimum soffit elevation for a proposed structure to provide adequate clearance above the estimated high ice elevation.
- Determining the magnitude and elevation of dynamic ice forces caused by ice floes crushing against pier(s).
- The maximum ice force would occur in a channel generally at or below the bank-full condition. For flows greater than bank-full, ice sheets may move onto the flood plain and be stored, thereby resulting in a reduced ice build-up.
- Designing counter measures to resist or reduce ice forces on a proposed structure.

The following method (Figure 5.24) may be used to estimate the design high ice conditions at a site:

1. Establish a typical channel and flood plain cross section for the proposed site.

2. For this section and the river reach, determine the representative bank-full and flood plain elevation by examining the site data.

   Determine the flood plain width for storage of flowing ice. If a flood plain width greater than two times the channel width exists on each side of the channel, the flood plain may be considered to be sufficient to provide for ice storage. This storage would be available for ice runs at elevations above top-of-bank as flow conveyance increases and ice floes spread over the flood plain.

3. Using channel characteristics (invert slope, S_o; Manning's roughness coefficient, n and cross section dimensions), determine the discharge and velocity for the bank-full stage, using Manning's equation.

   To start, take the elevation of flowing ice at bank-full level, assuming that the flood plain above will provide for a reasonable ice storage for ice flowing at elevations higher than bank-full. If the flood plain width is too small to provide for ice storage, try higher elevations where adequate flood plain storage for ice may be available.

   If historic data relevant to the site are not available, take this as a design value, recognizing that this estimate of high ice may be conservative. This may not be true for ice jam situations as the controlling stream section may be elsewhere in the reach.

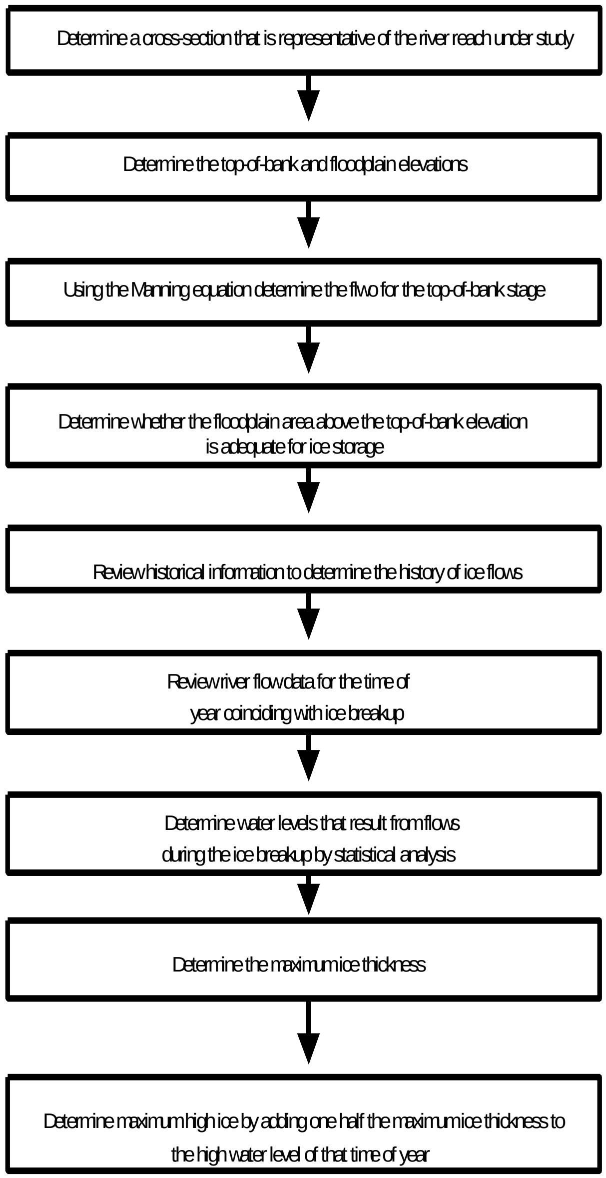
4. If additional information, such as freeze-up and breakup dates, historical high ice elevations are available, it should be used to help verify the above estimates.

   · From historical data provided by sources such as Environment Canada, determine the breakup dates and the corresponding discharges. If sufficient data is available, a frequency analysis may be carried out to determine the extreme (design) return period flows for the typical ice breakup period for the reach.
   · Estimate the stage for the extreme flow during breakup using the channel characteristics in the vicinity of the site, and Manning's equation (8.66).
   · With the above, compare and verify the estimate in Step 3. Also, check against known high ice elevation(s) from the past and estimate a design high ice elevation.

   Any historical information regarding existing bridges on the stream in close proximity may be used to further verify the above ice estimates provided that the river characteristics are similar.

The above method is applicable for estimating flowing ice conditions not associated with an ice jam and is illustrated in Design Example 5.8. Depending on the size of the stream channel and floodplain, more than one ice elevation and flowing ice situations "a" to "d" may be specified for pier design.

Figure 5.24: Estimating High Ice Conditions - Flow Diagram



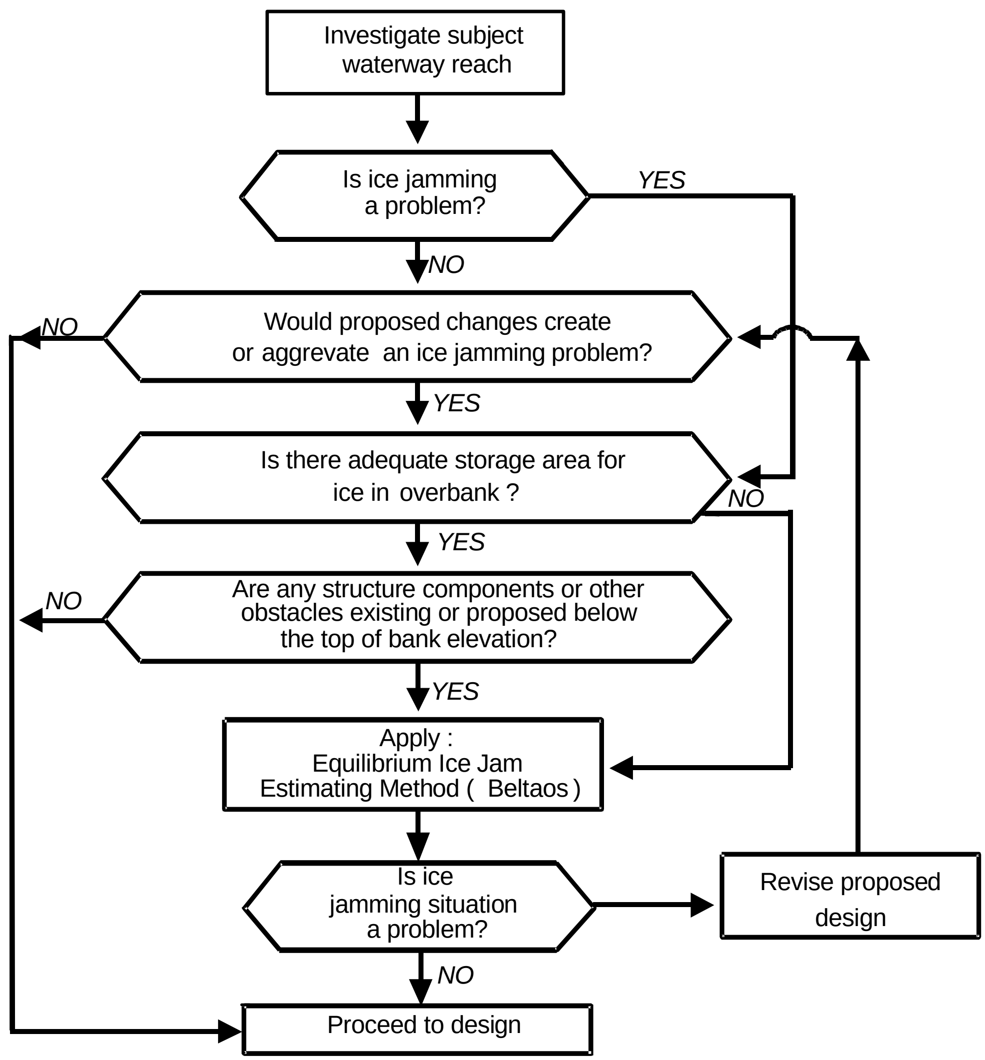
Assessing Ice Jam Condition

Assessing and analysing the risks of ice jams is complex and imprecise. A reach of a river may be periodically subjected to ice jams while other reaches of the same river may experience free flowing ice conditions. Ice jams are a local phenomenon and information on ice jams for one reach cannot necessarily be transposed to another reach.

Analytical techniques applicable to ice jams are much less developed compared with the hydraulics of open water flow, therefore, there is a greater need for site data to project and verify predictions.

The height of a potential ice jam may be estimated using the Equilibrium Ice Jam Estimating Method (Figure 5.25), outlined below:

Figure 5.25: Assessing Ice Jam Condition



Equilibrium Ice Jam Estimating Method

1. From a field visit and survey, determine channel cross sections and average height of bank(s) at which the channel will flow before ice and water flow moves over the flood plain(s).
2. Determine the typical channel width, W, and local invert slope, S, using contour plans or field surveys.
3. Determine the typical timing for breakup and ice conditions from historical records, archives or from long-term local residents.
4. From stream flow records (or by transposing information from neighbouring streams), estimate return-period flows for the time-of-year of ice break-up.
5. Using the hydraulic characteristics of the stream and the Manning equation, estimate the stage and width of flow for the discharge determined at step 4 above. Determine the discharge per unit width, q (m^2/s or m^3/s per m of width). Acceleration due to gravity, g = 9.81 m/s^2.

   Calculate $E = (q/gS)^{1/3} / WS$. (5.21)

6. Using Design Chart 5.21 (Beltaos, 1983), a dimensionless rating curve, determine equilibrium jam height, H, as:

   · Enter the Design Chart, with the value of E. From this rating curve read the corresponding value of N (= H/WS). (Eq. 5.22, Design Chart 5.20)
   · From the value of N estimated, determine H for the known values of W and S (step 2).

7. Verify the value of H with the channel section and the typical height-of-bank determined from field data.
8. If the channel is found to be prone to flowing ice conditions and actual channel depth is equal to or greater than H, an ice jam up to an equilibrium height, H, will likely develop. The height, H, would therefore be taken as the design value. If the height of the existing bank is lower and the flood plain is wide enough to store ice, the ice jam will likely not occur to an equilibrium stage. For a flood plain to be wide enough to store ice sheets, the flood plain width on each side should be at least two times the typical channel width .

The above estimates of ice jam height should be complemented, where possible, with a field investigation of historical ice jam information if available. Ice marks or scars on trees, banks or structures may provide clues.

An application of this method to assess the ice jam situation at a site is illustrated in Design Example 5.8.

Design Example 5.8
Estimating Flowing Ice Condition

Required

Estimate flowing ice conditions for design of a proposed multi-span bridge over a river channel.

Given

Past records show that this river is subjected to freeze-up every winter and ice break-up occurs usually in early spring. Interviews with local residents indicate that there have been heavy ice runs during break-up and a limited potential exists for ice jamming.

The channel cross section is approximately 250 m wide with steep banks to a height of 5 m. Over the banks, the floodplain is generally flat, cleared pasture, extending approximately 700 m transversely on each side. The river course in the vicinity, is fairly straight with a longitudinal slope averaging 0.0002 m/m.

The 100 year flow rate for the months of February and March, the time of spring ice break-up, was estimated to be 1500 m^3/s by statistical analysis of streamflow gauge records (Reference: Chapter 8). The water level for this event was 6.0 m above the river bed.

Flowing Ice Condition

A review of the history of this river channel has indicated heavy ice runs as ice typically breaks up and clears in February and March every year usually coincident with high runoff. The investigation has further indicated that large sheets of competent ice with thickness of up to 1.0 m were observed during past ice runs.

Using the Manning equation (Equation 8.66), estimate the flow depth with the 100 year flow (for February and March) of 1500 m^3/s. The result is a depth of approximately 5.5 m (elevation 105.5 m). For design purposes, it is postulated that large sheets of ice with a thickness of 1.0 m would break-up and flow downstream. Conservatively assuming 50 percent submergence, the top of the flowing ice will be at elevation 106.0 m. This is assumed to be the worst case scenario for the flowing ice condition.

The design flowing ice condition is assessed to be Situation "d" (*Ontario Highway Bridge Design Code*, 1991), considering that break-up may occur when the ice temperature is below freezing and the ice usually clears in large sheets which are sound and competent.

Potential for Ice Jam

The potential for ice jam formation is estimated using the Equilibrium Ice Jam Estimating Method. This method provides an estimate of the height of a fully developed ice jam, which may then be compared with the actual site conditions to determine if such a jam could actually occur.

River channel width, w = 250 m
Longitudinal slope, S = 0.0002
Acceleration due to gravity, g = 9.81 m/s^2

Therefore;
Discharge per unit width, q = Q / w
= 1500 m^3/s / 250 m
= 6.0 m^2/s

Estimate E.

$$E = (q^2 / (g * S))^{0.333} / (w * S) \qquad (5.21)$$
$$= (6.0^2 / (9.81 * 0.0002))^{0.333} / (250 * 0.0002)$$
$$= 510$$

Determine N.

N = 170 (Design Chart 5.20)

Solve for H.

$$H = N * w * S \qquad (5.22)$$
$$= 170 * 250 \text{ m} * 0.0002$$
$$= 8.5 \text{ m}$$

The estimated depth of a fully developed ice jam in this river is 8.5 m. As the banks are 5 m high, this ice jam will not fully develop. Ice floes will tend to move into the overbank areas when the top-of-bank elevations are significantly exceeded. Both flood plains should provide adequate space for ice storage. Therefore, the maximum stage for the flowing ice condition should not exceed elevation 106.0 m.

Conclusion

In summary, the design of this proposed structure should incorporate the following design ice condition:

- Maximum ice thicknesses of 1.0 m, which may be mainly composed of competent black ice at elevation 106.0 m.
- The flowing ice condition is characterised by Situation 'd'.

Debris Flow

Rivers and streams, when flooded, may carry debris (Figures 5.26 and 5.27). The bulk of debris consists of tree material and other vegetation that is floated by high flows and/or uprooted by erosion undercutting the stream banks and then carried downstream by the flow.

Some debris, resulting from channel erosion or slumping of banks, may not float and travel downstream but may obstruct the flow path. The amount of debris flow will be dependent on the carrying capacity of the channel. Large debris, such as logs, are generally carried by streams with a relatively wide channel or flood plain and greater velocity of flow. Narrow channels, sharp bends, channel bars or islands and waterway constrictions, such as bridges or culverts may cause deposition of floating debris.

Factors Affecting Debris Flow

- Stream channel width; a narrow channel width would not permit large debris, such as trees, to flow downstream. However, during high flows, debris may be carried by flow over flood plains. Debris with large dimensions relative to the width and depth of the flow path will tend not to move downstream. Such large debris will tend to block the watercourse and may trap smaller debris and cause backwater.
- Type(s) of soils in stream banks and their susceptibility to erosion. Erosion may cause vegetation to be uprooted.
- Type(s) of vegetation growth on stream banks, that are susceptible to uprooting or dislodging from the forces of flow and/or erosion.
- Stream geometry; a meandering creek usually has the capacity to carry only smaller vegetation, such as willows or tree branches. Curves and constrictions in a stream channel will tend to trap flowing debris.
- Average channel slope; a steeper slope would tend to result in faster flow velocities that would provide greater force to move debris. However, a shallower slope would tend to result in greater flow depths, hence allowing larger debris to float freely downstream.

The characteristics of a debris flow situation for a particular river is best evaluated by a site or area visit and interviews with local residents or persons familiar with the area. Local highway or road maintenance staff may be familiar with debris problems at existing structures crossing the river of interest. Debris may be present on stream banks or trapped at constrictions, such as bridge or culvert crossings.

Figure 5.26: Severe Debris Problem at a Bridge Crossing

Figure 5.27: Bridge Opening Obstructed by Logs

Impact of Debris

Flowing debris results in the following types of adverse impacts to channel and water crossings.

Reduces Conveyance Capacity

Floating debris may cause blockage and therefore reduce the conveyance capacity of a water crossing. This may increase upstream flooding and accelerate channel erosion in the vicinity and downstream.

Structural Problems

Flowing debris may cause blockage and excessive erosion in the vicinity of a structure, including abrasion of embankments. Excessive erosion may result in undermining of structure foundations. Debris blockage may also transfer forces of flow to a bridge superstructure, tending to dislodge it.

Controlling Debris

As discussed above, by studying the watershed and stream channel, it may be possible to assess the potential for debris flow at a site of interest. However, the occurrence and severity of debris relative to a flood event is hard to predict. In practice, the problem of debris is generally handled by avoiding difficult sites, by providing flow deflectors to facilitate the passage of debris or by providing a larger waterway opening than would otherwise be required. Debris racks for culverts have been used with a limited success on streams with light to mild debris as they require periodic cleaning.

The choice should generally be made during preliminary design stage rather than the detail design stage. (Reference: Chapter 3).

Remedial Erosion Control Measures

Table 5.7 presents remedial erosion control measures that should be based on an accurate description of the problem. Figure 5.28 presents various erosion indicators. Watershed data required to define the causes of erosion include:

- past, present and projected land uses;
- prior channel modifications;
- past stability problems;
- previous treatment measures;
- flow rates, regime, depths and velocities for a range of flow rates; and
- transported sediments.

Specific site data required to define erosion problems include:

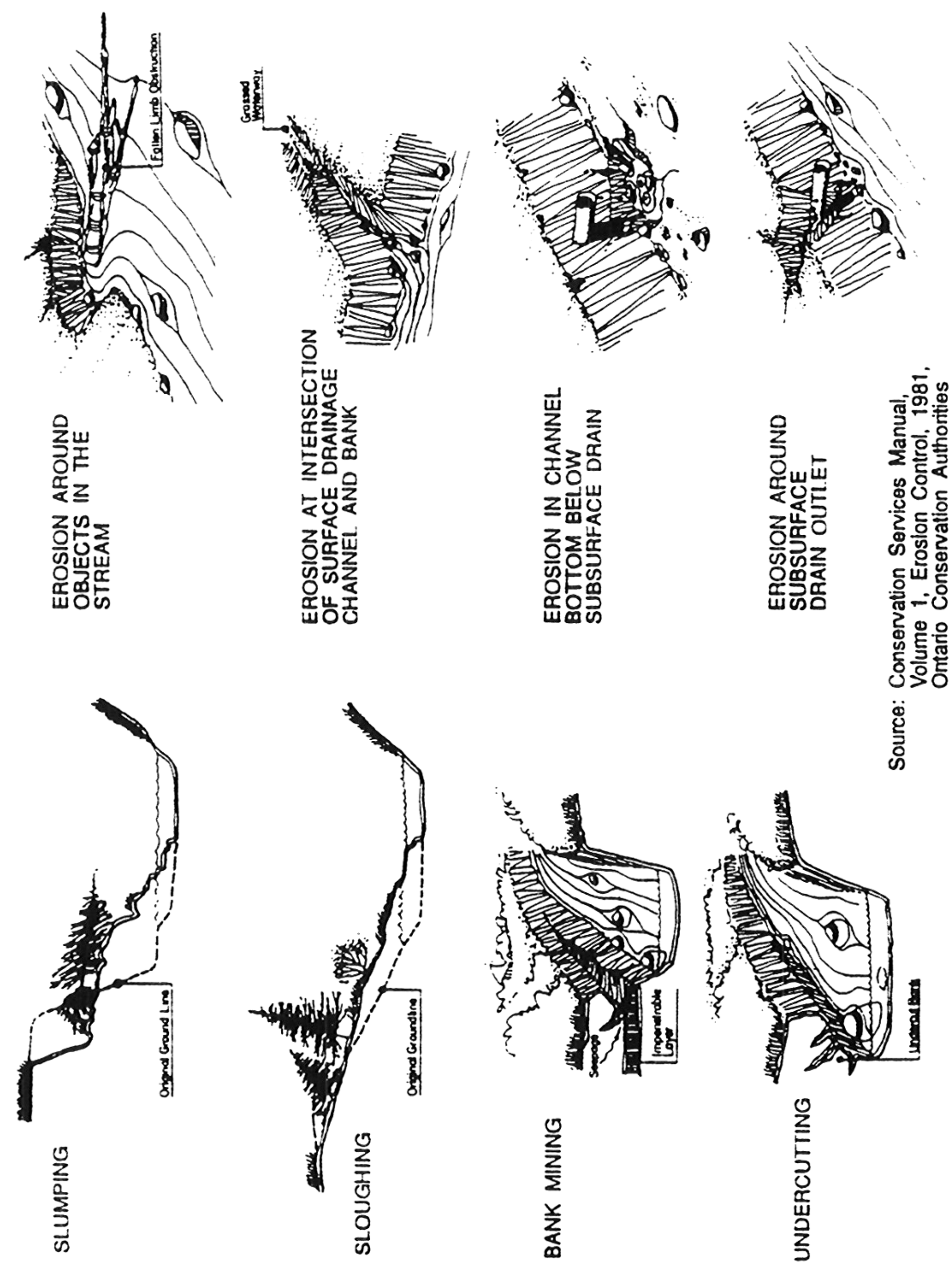
- cross section surveys;
- stream pattern and alignment;
- channel gradient;
- types and magnitude of obstructions;
- bank vegetation, soil and geotechnical
- water quality; and
- existing aquatic/terrestrial habitats.

Table 5.7: Remedial Erosion Control Measures

Indicator	Problem	Cause	Typical Remedial Measures
• sloughing • slumping	• stability	• bank steepness • excess soil moisture and soil load	• grade and cover • retaining wall • bank drainage
• bank "mining"	• seepage	• seepage out of bank above an impermeable layer	• bank drainage
• rilling	• surface runoff	• surface runoff over bank	• cover • bufferstrip • runoff diversion • drop structure
		• roadway adjacent activities and erosion problems	• cover • runoff diversion • drop structure
• undercutting • erosion around objects in the stream	• stream flow	• high flow velocity	• cover
		• localized scour around structures in the stream	• cover
		• channel curve	• cover • retaining wall • groynes
		• steep channel • localized scour around obstructions in the stream	• major channel works • major channel works
• erosion at intersection of surface drainage channel and bank • erosion around subsurface drain outlet erosion in channel bottom below subsurface drain outlet	• drainage outlet	• poorly located or constructed surface drain outlet	• runoff diversion • drop structure
		• poorly constructed subsurface drainage outlet	• proper outlets
• turbid water • sediment deposits	• sediment transport	• upstream activities and erosion problems	• sediment traps
• gullying • field dissection	• gully erosion	• concentrated surface flow	• grade and cover • runoff diversion • check dams • fill and regrade • drop structures
		• seepage out of bank	• bank drainage

(MTO, 1992)

Figure 5.28: Erosion Indicators



Stream Channel Sections

In general, stream channel modification and open stream channel cross sections may either be natural or artificial. Generally, natural sections are of irregular shape and tend to suit geomorphic processes, while artificial or prismatic sections are of rectangular, trapezoidal and triangular cross-section and do not vary with distance along the stream channel. Several other section shapes are used for special circumstances.

Artificial/Composite Sections

Usually, composite stream channels have an upper and a lower section that may have different shapes and lining materials. The lower section is used to convey frequent runoff events while the upper section conveys the larger infrequent events.

A composite (double) trapezoidal section can be used to solve many open channel design problems. As an example, a triangular section can be approximated by a trapezoidal section with a bottom width equal to zero, while a rectangular section can be approximated by a trapezoid with vertical side slopes. Figure 5.29 shows a composite trapezoidal section may be used to approximate most combinations of rectangular, triangular, trapezoidal and natural cross sections.

Figure 5.29: Composite Channel Definition Sketch

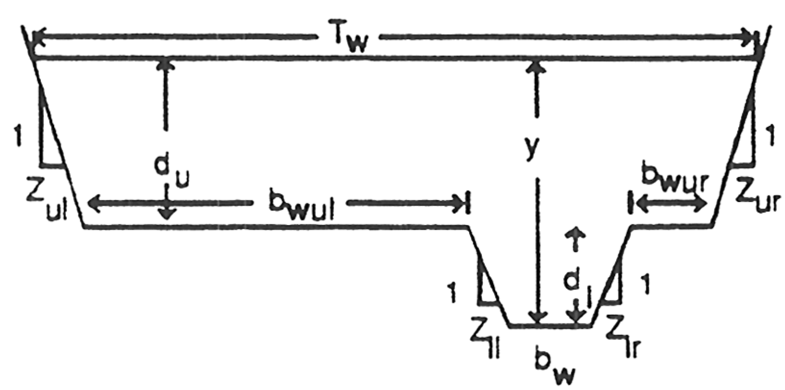
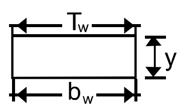
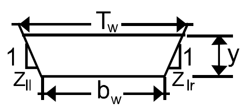
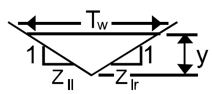


Figure 5.30: Flow Parameters

Section	Area A	Wetted Perimeter P	Hydraulic Radius R	Top Width T_w	Hydraulic Mean Depth $\bar{y}_m$
	$b_w y$	$b_w + 2y$	A / P	b_w	A / T_w
	$\frac{y^2 Z_{ll}}{2} + \frac{y^2 Z_{lr}}{2} + y b_w$	$y\sqrt{1+Z_{ll}^2} + y\sqrt{1+Z_{lr}^2} + b_w$	A / P	$b_w + yZ_{ll} + yZ_{lr}$	A / T_w
	$\frac{y^2 Z_{ll}}{2} + \frac{y^2 Z_{lr}}{2}$	$y\sqrt{1+Z_{ll}^2} + y\sqrt{1+Z_{lr}^2}$	A / P	$yZ_{ll} + yZ_{lr}$	A / T_w

Legend

- A = Flow area, m^2
- b_w = Channel bottom width, m
- R = Hydraulic radius, m
- T_w = Top width of flow, m
- P = Wetted perimeter, m
- y = Flow depth, m
- $\bar{y}_m$ = Hydraulic mean depth, m
- Z_{ll} = Lower left side slope, H:V
- Z_{lr} = Lower right side slope, H:V

Natural Channel Sections

The success of natural channel design is dependent on several factors including the fluvial processes that may alter the physical and biological characteristics of the watercourse. Alterations include shape, location, width, depth, length, sinuosity, and water quality (sediment load and temperature). The alterations are the result of changes in the watershed geology, topography, soils, vegetation and precipitation. To be successful, the design of natural watercourses must incorporate fluvial processes.

Predicting alterations in natural watercourses is extremely difficult. Considerable judgement must be used when designing, locating and sizing stable natural channels. Chapter 9, and the MNR document *Natural Channel Systems: An Approach to Management and Design, Draft June 1994,* is referenced for details on pools, riffles, point bars, as well as other aspects of stream channel morphology.

For design purposes, the following guidelines may be applied to create pool/riffle areas when specific data is not available.

- Pool and riffles should be created at intervals of approximately 5 to 7 channel widths.
- Instream channel devices should be located and constructed to minimize upstream channel backwater during large stream channel flow events.
- Instream channel devices should be:
    - constructed to freely pass logs and debris;
    - able to withstand large stream channel flow events; and
    - constructed of natural materials.
- Construction timing should avoid spawning and incubation periods.
- Stream channel banks should be well protected if instream channel devices are used.
- In cases where the riffles are to be dynamic and self-sustaining, they should be constructed from natural stream channel gravel with a size distribution typical of the existing bed material. Pools should have a minimum low-water depth of 0.3 m and should be constructed in meanders. Riffle material should not project above the bed by more than 0.3 to 0.5 m. Asymmetric cross-sections that approximate natural cross-sections area are recommended. In general, pools should be located on the outside of the watercourse bends. Individual pools or riffles should not be longer than three channel widths or shorter than one channel width.

Riffles can be constructed using smooth, rounded rock to minimize potential injury to fish. They should be located in straight sections where the thalweg crosses over. Riffles can double as grade control structures and should not project more than 0.3 to 0.5 m above the sub-channel invert.

The Design or Selection of a Suitable Stream Channel Section

The design of open channel sections is an iterative process as many combinations of stream channel shapes, stream channel slopes and lining materials may satisfy the project criteria. The design usually begins with the estimation of a stream channel shape and size, which is then followed by the selection a lining material that can withstand the flow condition. The key parameters are usually flow depth, flow velocity or tractive force. The allowable flow depth and flow velocities are a function of the original design goals.

A preferred design is a composite cross section which achieves desired functional purposes. For example, the lowest portion of the stream channel can be designed to convey the base flow or low flow, with a sufficient depth that can suit aesthetic and aquatic purposes. The capacity of the low flow channel should be approximately equal to an event with a higher frequency of occurrence (e.g. 1.5 to 2 years). The more frequent flows are conveyed in the portion of the stream channel located between the low flow and bank-full levels (e.g. return periods typically ranging from the 2 year to 25 year). The flood plain, located above the bank-full level, should safely convey the more severe flood events (e.g. 25 year return period to regulatory flood).

Depth is always calculated to ensure flow is contained within the stream channel corridor. Velocity or tractive force is used to determine if the lining material will be stable or non-erosive under design conditions.

The relative change in depth and velocity for a given change in slope, stream channel size and roughness is as follows:

- slope increases => depth decreases, velocity increases;
- roughness increases => velocity decreases;
- flow top width increases => depth decreases, velocity increases; and
- side slope increases => velocity increases.

Stream channels should preferably be designed to accommodate subcritical flow. Flow depths within 10% of critical depth should be avoided. For a discussion of different types of flow, refer to Chapter 8.

Stream Channel Lining Materials

Many materials have been used to permanently line open channels. Lining materials are classified as either flexible or rigid. Examples of flexible lining materials include vegetal, riprap/rock, gabions, armour stone and articulated concrete block. An example of a rigid lining is concrete. Soil bioengineering, lining filters (geotextile is considered to be a lining filter) and buffer strips are also included within.

Information on the following lining materials can be found in the Fact Sheets shown in Appendix 5C.

Table 5.8: Stream Channel Lining Materials

Lining	Stable Flow Velocities
Vegetal	Low
Riprap/rock	↓
Gabion	↓
Concrete block	↓
Armour stone	↓
Concrete	High

The Fact Sheets (Appendix 5C), identify typical stream channel shapes, criteria, guidelines, detailing, specification requirements, design steps, construction steps, advantages and disadvantages for each of the lining materials. Stream channel lining materials are presented in Table 5.8. This section does not address fish passage and fish habitat concerns. For such cases, the use of vegetation and/or natural bed materials is desirable.

Soil Bioengineering

Soil bioengineering requires extensive site assessments of soil conditions, plant species and the erosion mechanism. The implementation of any soil bioengineering project should involve an inter-disciplinary team approach consisting of specialists in the geotechnical, horticultural, engineering, drainage, environmental, and landscape architecture fields. Technical and ecological parameters must be considered before the plant species and soil bioengineering systems are selected. Critical factors also include timing and the method of installation.

Table 5.9: Channel Lining Summary

Type of Lining	Advantages	Disadvantages	Possible Maintenance
Grass	• inexpensive • aesthetically pleasing	• possibility of erosion prior to establishment of vegetation • limited riparian habitat value	• regular mowing • immediate action to repair waterway of any bare spots
Dumped Riprap/Rock	• construction skills widely available • aesthetically pleasing • flexible • locally available • tolerates lateral seepage	• selection of stone size, distribution and shape vital to stability and erosion protection	• removal of woody vegetation • replacement of lost riprap/rock
Hand Placed Riprap/Rock	• neat appearance	• labour intensive • loss of riprap/rock caused by children throwing the material • tendency to be destroyed by high flow velocity	• replacement of lost riprap/rock • removal of woody vegetation
Grouted Riprap/Rock	• acceptable appearance • minimum vandalism problem	• easily undermined • lack of flexibility to adjust to movement	• prompt repair to any cracking and undermining required
Gabion Mattress/Gabions	• flexible • smaller stone size than riprap/rock	• expensive • tendency of movement by water at upstream end • possible occurrence of outflanking behind gabions • wires deteriorate and break • difficult to install in deep water	• regular inspection of lining condition • prompt repair to any damages, particularly wire meshes
Articulated Concrete Mats	• acceptable appearance • allows some vegetation • lower friction than riprap/rock • life expectancy 10-50 years • construction not hindered by access	• expensive • needs good subgrade treatment • does not control brush • difficult to replace individual units	• ice damage • removal of accumulated debris
Concrete	• hydraulically efficient • effective erosion control method • minimum maintenance required	• expensive • not aesthetically pleasing • susceptible to hydraulic uplift • low habitat diversity	• repair lining damages due to hydrostatic uplift and frost • removal of accumulated debris
Armour Stone	• maintenance and repairs easily completed • life expectancy 10-20 years	• requires heavy construction equipment and good access • does not stabilize against large slope failure	

(MTO, 1992)

Table 5.9: Channel Lining Summary (cont'd)

Type of Lining	Advantages	Disadvantages	Possible Maintenance
Soil Bioengineering	• natural appearance • provides wildlife habitat • flexible does not deteriorate • self supporting and generating • used in combination with other methods • not capital intensive	• soil materials required for stability may not be adequate for plant growth • vulnerable to trampling, drought grazing, foxing and pests • utilize plant materials that may not be abundant or locally available • installation does not coincide with construction season or high winds	

Note: Geotextile is considered to be a filter

The design requires expertise in soil mechanics, hydrology/hydraulics, groundwater, environmental planning, landscaping and vegetation management. Hydrologic and hydraulic information should include duration of flooding, peak flow rates, maximum flow velocities, tractive forces, overland flow patterns, magnitudes and associated frequencies of occurrence. A more detailed discussion on soil bioengineering may be found in Chapter 10.

Lining Filters

Water can enter the watercourse through the stream channel banks (e.g. groundwater movement) carrying soil with it. Any voids left by the soil particle movement will increase in size until the lining collapses. To prevent collapse, a filter can be placed under the channel lining to prevent loss of soil particles and the collapse of the lining material.

The filter material should be designed to allow water to move through the material while at the same time preventing the migration of the soil particles of the underlying material. Granular and synthetic filter blankets have been used to successfully prevent lining failures.The following criteria have been used to design granular filter blankets.

$$\frac{D_{15\ Filter}}{D_{85\ Base}} < 5 < \frac{D_{15\ Filter}}{D_{15\ Base}} < 40 \quad \text{and} \quad \frac{D_{85\ Filter}}{D_{85\ Base}} < 40 \qquad (5.23)$$

If a single layer granular filter blanket is not sufficient then several layers must be used.

Geotextile filters have been widely used to replace the sometimes costly process of providing granular filter layers. The basic principles of preventing fine material from passing through the filter while allowing the passage of water still apply to geotextile. The selection of geotextile filters is based on tensile strength, abrasion resistance, stretch, loss of strength when wet, moisture regain, filtration opening size (F.O.S) and equivalent opening area.

When D_{50} of the granular material being placed adjacent to any geotextile filter, will be less than the 80 mm sieve, the following criteria can be applied.

- The F.O.S. range for the geotextile is related to the D_{85} of the soil to be protected (D_{85} is the particle diameter corresponding to 85% finer on the grain size curve). The upper F.O.S. limit should be equal to the D_{85} of the soil and the lower F.O.S limit should be equal to half of the upper limit.
- If more than one soil type is present then the F.O.S. of the geotextile should be based on the D_{85} of the finest underlying soil type. The specification of two geotextiles with different opening sizes should also be considered.

Buffer Zones

Buffer zones can be established to protect existing or proposed vegetation adjacent to watercourses. Riparian vegetation is closely linked to the health of the aquatic habitat.

Vegetation provides shade, can stabilize stream channel banks and can favourably influence aquatic life. Although buffer strips are extremely important to aquatic life, their effectiveness may not be evident because of the surface water quality being conveyed by the watercourse.

The Design and Selection of Lining Material

Selection of a lining material is usually a function of material stability under design flow conditions. Generally, flow velocity or tractive force is used to determine the stability of a lining material. Maximum permissible velocities or maximum permissive tractive forces have been established for different lining materials (Design Charts 2.22 and 2.23). The values are established by several methods including trial and error, physical modelling and theoretical equations. The designer is responsible for selecting the most appropriate values.

The maximum permissible velocity method considers only channel slope while the tractive force method is usually more accurate as it intrinsically considers the effect of hydraulic radius and channel shape. For details of both the permissible velocity and tractive force methods refer to the Stream Channel Erosion Analysis section. Design Chart 2.24, shows the relationship between tractive force and velocity for a specific roughness coefficient and channel slope.

Where hydraulic conditions are influenced by upstream channel or downstream channel levels, design should be carried out for both conditions with and without downstream channel influences.

A guide to designing stream channel lining protection is presented below:

- Vegetal lining, where appropriate to resist stream channel erosion, should be preferred over rock lining because of its natural and aesthetic appeal.
- The layer thickness for a riprap blanket should be at least 1.0 * D_{100} or 1.5 * D_{50}. Quite often, to add a safety factor, 2.0 * D_{50} may be used for layer thickness (U.S. Army Corps of Engineers, 1970).
- Gabion baskets and articulate blocks connected with cable may be used as an alternative to riprap. However in ice and debris prone situations, they should be avoided as they may be damaged by abrasion. Typical layouts for gabion slope protection and articulate blocks are shown in the Fact Sheets (Appendix 5C).
- Armour stone blocks may be used as an alternate means of protection for embankments and stream channel beds. Refer to the Fact Sheets (Appendix 5C).
- Cast-in-place concrete has been used in the past as stream channel lining for erosion protection (Refer to the Fact Sheets in Appendix 5C). This use is discouraged for several reasons. The concrete often breaks up after some years due to differential settlement of the underlying soil. Maintenance cost is high. Concrete is relatively impermeable and harsh to natural drainage and the environment. It's smooth surface causes an increase in the conveyance capacity of a stream channel to the possible detriment of downstream lands.
- Provide a toe at the end/edge of a lining and protective cover to key the lining into the natural ground to reduce the vulnerability of it being undermined due to scour or erosion.
- For scour protection, refer to the section on Scour.

Stream Channel Bends, Meanders and Alignment

Stream Channel Bends

Erosion often occurs at stream channel bends which are typically exposed to higher flow velocities on the outside of the bend than straight stream channel reaches. During the design process, tractive forces should be carefully reviewed to ensure that the selected lining material can sustain these higher flow velocities. Figure 5.31 identifies typical high tractive force zones in stream channel bends and the approximate location and length of stream channel protection works to prevent erosion.

Meanders

Watercourse enhancement or stream channel design may include the creation or rehabilitation of meanders for the support of the pool/riffle regime. The constructed meanders may be very similar to the meander characteristics of adjacent reaches of the natural watercourse. Meander characteristics include length, amplitude, radius of curvature and cross-section shape. Meander characteristics (refer to Chapter 9) can be determined from aerial photographs, historical maps, and site investigations along the watercourse in both the upstream and downstream directions. The meander cross section should be asymmetric with steep banks on the outside bend and would be constant throughout the meander.

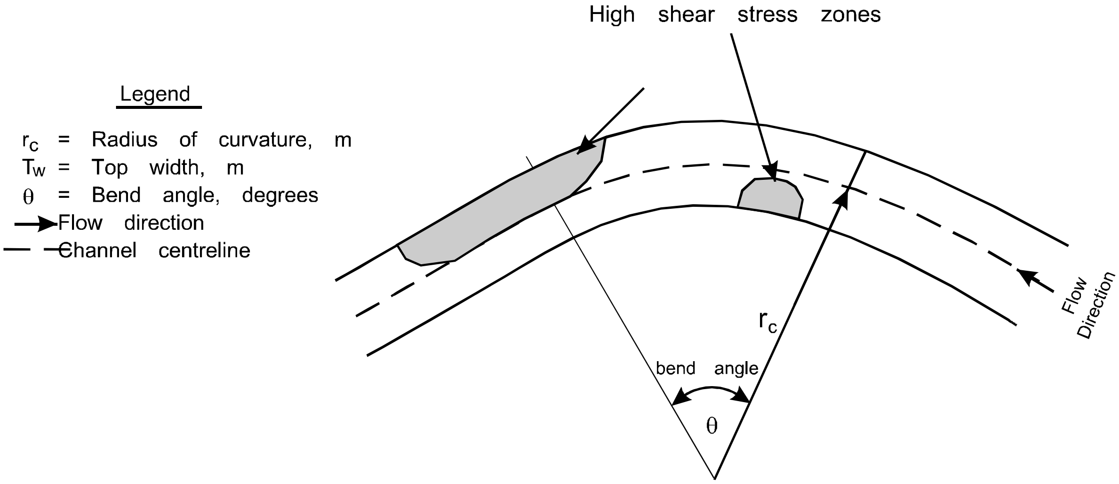
Meanders can be either excavated within a watercourse or can be induced using deflectors on alternating banks. The following guidelines can be used to design meanders when historical or site data are not available:

- meander length varies about 7 to 10 times the stream channel width as measured in a straight line;
- the length between identical points on the meander wave varies from 10 to 16 times the stream channel width; and
- successive crossovers (2 per meander) are spaced at 5 to 7 times the stream channel width to reflect the natural occurring interval of pools and riffles.

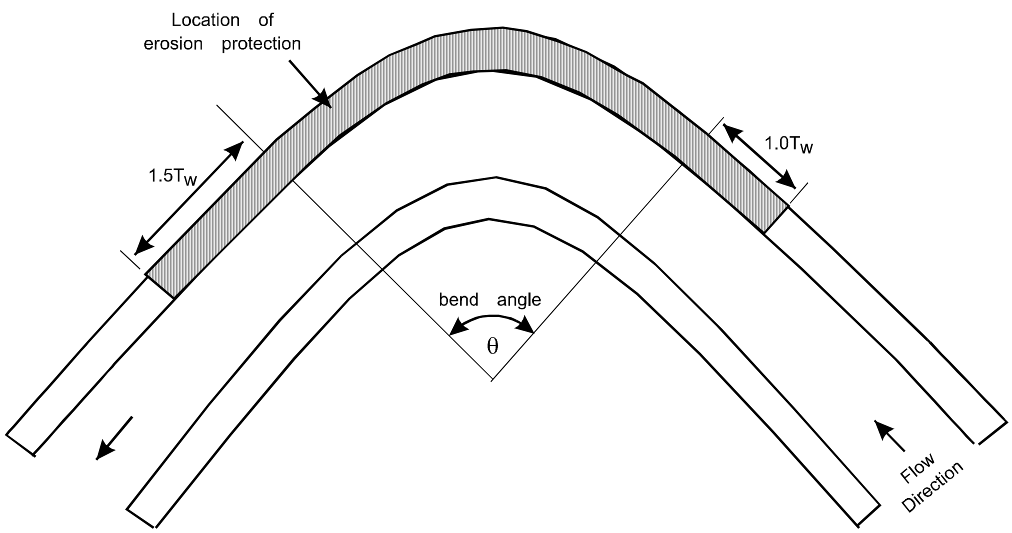
The following procedure can be used to design meanders:

- field surveys and aerial photographs should be used to determine existing meander characteristics (wavelength, amplitude or width, and stream channel length);
- layout new alignment considering topography and geology;
- incorporate pool/riffle sequence with uneven stream channel gradients;

Figure 5.31: Channel Bend Protection



Source: After Shear Stress Distribution in Stable Channels Bends, 1979, by M.A. Nouh and R.D. Townsend.



Source: After The Streambed Erosion Control Evaluation and Demonstration Act of 1974, Section 32, Public Laws 93-251: Final Report to Congress, "Main Report and Appendices A through H, December 1981, by U.S. Army Corps of Engineers.

- establish meander geometry, cross section shape and transition between pool and riffle (field surveys should determine the shape and dimensions of natural meanders);
- design substream channel; and
- determine lining material for outside of meander banks (i.e., riprap/rock).

Alignment

Factors influencing stream channel alignment include the following:

- roadway alignment;
- significant or sensitive environmental features;
- receiving water bodies;
- physical landscape features (i.e. rock outcrops);
- watershed boundaries;
- natural stream channel regime;
- impact on adjacent or downstream channel land owners; and
- political boundaries, municipalities, etc.

Stream channels with small gradients usually exhibit low flow velocities. During low flows, sedimentation and ponding will occur with reductions in stream channel capacity. Stream channels with gradients between 0.1 and 0.5% are recommended to minimize maintenance requirements. However, stream channels with gradients less than 0.1% are difficult to construct.

Stream Channel Erosion Analysis Methods

The dimensions of erodible stream channels are usually determined by applying the Manning equation (Eq. 8.66) as well as equations related to the bed material and sediment load. There are two commonly used theories upon which methods for the design of erodible channels are based:

- maximum permissible velocity; and
- maximum permissible tractive force.

Maximum Permissible Velocity

The maximum flow velocity that a channel can withstand without serious deformation depends on the bed material, flow depth, velocity, sediment load, channel alignment and vegetation. Typical maximum permissible velocities are given in Design Chart 2.17, for different lining materials. The design of stream channels is affected by the flow rate, Q, the longitudinal bed slope, S, and the type of bed material. Based on this information, estimates can be made of the Manning roughness coefficient, n, the side slope ratio, Z, and the maximum permissible flow velocity, V_{max}.

To find the depth of flow, y, in a prismatic channel for a particular flow rate, Q, Manning's equation is iterated. Computer programs, such as CHANDE, are available for this purpose. After obtaining the channel cross section shape, the Manning's roughness coefficient, n, and the longitudinal slope, S, a flow depth, y, is estimated. Then, flow cross-section area, A, and hydraulic radius, R, are estimated as shown below. Manning's equation is then solved for flow rate, Q. The depth, y, is reiterated until the resulting Q is close to that desired.

The flow cross section area, A, of a trapezoidal channel may be calculated as follows:

$$A = y * (b_w + Z * y) \tag{5.24}$$

where:

- A = flow cross section area, m^2
- y = flow depth, m
- b_w = channel bottom width, m
- Z = channel side slope ratio, horizontal run / vertical rise

The wetted perimeter, P, of a trapezoidal channel can be calculated as follows:

$$P = b_w + 2y * (l + Z^2)^{1/2} \tag{5.25}$$

where:

P = wetted perimeter, m;
b_w, y, Z as above

$$\text{Hydraulic Radius, } R = \frac{\text{Flow cross-section area, A}}{\text{Wetted Perimeter, P}}$$

The maximum hydraulic radius as a function of the maximum permissible velocity can be determined using the Manning equation (rearranged),

$$R_{max} = (V_{max} * n / S^{1/2})^{3/2} \tag{5.26}$$

where:
- R_{max} = maximum hydraulic radius, m
- V_{max} = maximum permissible flow velocity, m/s
- n = Manning's roughness coefficient
- S = channel bed slope, m/m.

The channel must be proportioned, such that hydraulic radius, R, does not exceed R_{max}. Therefore, b_w, y and Z must be such that $R \le R_{max}$.

Maximum Permissible Tractive Force

Tractive force is defined as the shear force exerted by the flow on to the wetted channel surfaces. Tractive stress (N/m^2) is the tractive force (N) per unit area (m^2).

The tractive force concept can be used to design flexible channel linings. If the tractive force caused by the flow is greater than the resistive forces holding the material, then erosion occurs.

Resistive forces are defined in literature (Design Chart 2.25) for cohesive and non-cohesive materials. Cohesive materials, such as clay, have a bond between particles. For a particular particle size, cohesive soils are generally less susceptible to erosion. Non-cohesive soils consist of discrete particles such as sand, gravel and riprap materials.

Shear Forces

As shown on Figure 5.32, the tractive stress varies along the bed and sides of a channel.
For design purposes, the designer needs to know the maximum tractive stress along the channel bed and banks. The following equation can be used to determine the maximum tractive stress:

$$\tau_{b\ max} = K_b * \gamma * R * S \tag{5.27}$$

where:

$\tau_{b\ max}$ = maximum tractive bed stress, N/m^2
K_b = tractive force (bed) coefficient (Design Chart 2.11)
γ = unit weight of water, 9810 N/m^3
R = hydraulic radius, m
S = channel slope, m/m

The maximum tractive stress along the side of a trapezoidal channel can be calculated using the following equation:

$$\tau_{s\ max} = K_{bk} * \gamma * R * S \tag{5.28}$$

where:

$\tau_{s\ max}$ = maximum tractive bank stress, N/m^2
K_{bk} = tractive force (bank) coefficient (Design Chart 2.12)
γ, R, S as above

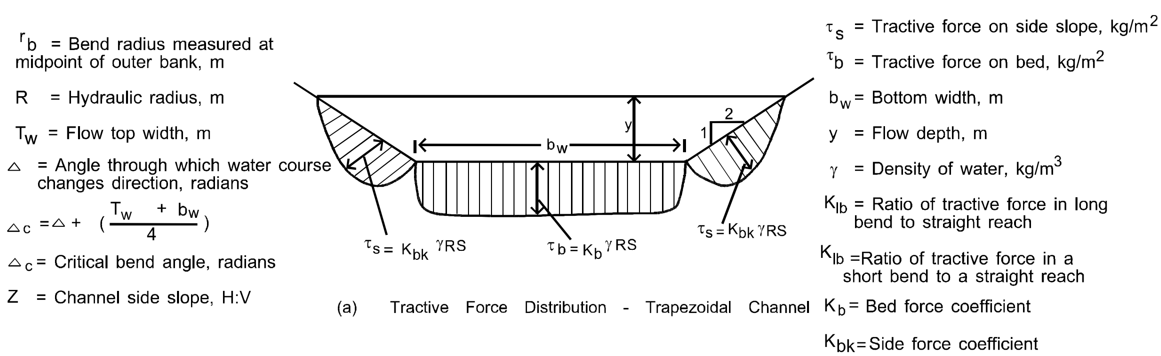
The tractive stress increases on the outside of bends due to the effects of centrifugal forces. The sharper the bend, the greater the increase in tractive stress. The maximum tractive stress in a bend is a function of the maximum tractive stress along the channel bed. The following equation can be used to determine the maximum bend tractive stress:

$$\tau_{bend\ max} = K_{bd} * \tau_{b\ max} \tag{5.29}$$

where:

$\tau_{bend\ max}$ = maximum tractive bend stress, N/m^2
$\tau_{b\ max}$ = maximum tractive bed stress, N/m^2
K_{bd} = tractive force (bend) coefficient (Design Charts 2.26 and 2.27)

Figure 5.32: Tractive Force Distribution



The following defines short versus long bends for use in Design Charts 2.26 and 2.27:

Δ = angle through which the watercourse changes direction
r_d = $r_b + [(T_w + b_w) / 4]$

where:
- r_b = bend radius measured at mid height of outer bank, m
- T_w = flow top width, m
- b_w = channel bottom width, m

Δ_c = Arc cosine $[r_b / r_d]$, the critical bend angle

if: $\Delta \le \Delta_c$; bend is short
if: $\Delta > \Delta_c$; bend is long

Cohesive Resistive Force

Very little information exists on resistive forces of cohesive materials. Design Chart 2.25, shows the permissible tractive stresses for different soils and levels of compaction.

Non-Cohesive Resistive Force

Many studies have been conducted to determine the critical shear stress of non-cohesive particles. The critical shear stress of particles on a channel bed can be determined using the following equation:

$$\tau_{cb} = \frac{W_s \tan\theta}{A_p} \qquad (5\text{-}30)$$

where:
- τ_{cb} = critical side shear stress of particles on channel bed, N/m^2
- W_s = weight of particle, N
- A_p = area of particle, m^2
- θ = bank angle

Research indicates the critical shear stress along the channel bed can be approximated using the following equation:

$$\tau_{cb} = 0.0642 * D_{50} \qquad (5.31)$$

where:
- τ_{cb} = critical side shear stress of particles on channel bed, kg/m^2,

D_{50} = median particle size, mm.

The critical shear stress of particles on a channel side slope is a function of the angle of repose of the material and the side slope of the bank in addition to the above parameters for a particle on a level bottom surface. The following equation can be used to determine the critical side shear stress:

$$\tau_{cs} = K_{sb} * \tau_{cb} \tag{5.32}$$

where:

τ_{cs} = critical side shear stress, N/m^2

K_{sb} = $(1 - \sin^2\theta/\sin^2\phi_s)^{0.5}$

where: θ = channel side slope angle

ϕ_s = angle of repose

τ_{cb} = critical shear stress of particles on the river bed, N/m^2

Erosion will occur if the tractive force generated by the flow velocity is greater than the resistive force of the lining material.

Design Example 5.9
Bank Protection Design

Required

Design bank protection for a stream channel that is eroding along its banks.

Given

- design flow, Q_{100} = 60.6 m^3/s
- Stream channel longitudinal water surface slope, S_o = 0.005
- Manning's coefficient, n = 0.035 for natural channel (Design Chart 2.01)
- Proposed stream channel cross-section
  - Bottom width, b_w = 2.0 m
  - Side slopes = 2:1 (Z=2; angle $\theta = 26.5^o$)
- In-situ soils: non-cohesive sand and gravel
  - D_{50} = 1.0 mm
  - Angle of repose, $\phi = 35^o$
  - Unit weight of water, γ = 9810 N/m^3

Analysis

Calculated Data, assuming a prismatic channel and reiterating Manning's equation: (reference Chapter 8):

- depth, y = 2.9 m
- flow cross-section area, A = 22.6 m^2
- wetted perimeter, P = 15.0 m
- flow velocity, V = 2.68 m/s
- Hydraulic radius, R = A/P = 1.51 m

Mean boundary shear stress, τ_o, for the existing stream channel

$$\tau_o = \gamma * R * S_o \tag{5.33}$$
$$= 9810\ N/m^3 * 1.51\ m * 0.005 = 74\ N/m^2$$

Stream channel bottom

$$\tau_b = K_b * \tau_o \tag{5.34}$$

where: K_b = 1.3 (input: b_w, y, Z) (Design Chart 2.11)

$\tau_b = 1.3 * 74\ N/m^2 = 96\ N/m^2$

Stream channel Sides

$\tau_s = K_{bk} * \tau_o$ (5.35)

where:

$K_{bk} = 1.4$ (input: b_w, y, Z) (Design Chart 2.12)

$\tau_s = 1.4 * 74\ N/m^2 = 104\ N/m^2$

Shear stress resistance, τ_{cb}

Existing stream channel bottom

$\tau_{cb} = 0.0642 * D_{50}$ (mm) (5.31)
$= 0.0642 * 1.0$ (mm)
$= 0.0642\ kg/m^2 = 0.63\ N/m^2$

Existing stream channel sides

$\tau_{cs} = K_{cs} * \tau_{cb}$ (5.36)

where: $K_{cs} = (1 - (\sin^2\theta / \sin^2\phi))^{0.5}$ (5.37 or Design Chart 2.14)
$= (1 - (\sin^2 26.5^o / \sin^2 35^o))^{0.5}$
$= 0.63$

where: sideslope angle $\theta = 26.5^o$
angle of repose of bank material, $\phi = 35^o$ (Design Chart 2.13)

The coefficient K_{cs} is less than or equal to 1 and represents the reduced shear stress resistance of particles resting on a slope compared to particles on a level surface.

$\tau_{cs} = K_{cs} * \tau_{cb}$ (5.36)
$= 0.63 * 0.0642$
$= 0.040\ kg/m^2 = 0.39\ N/m^2$

The stream channel at the subject site is fairly straight. If there was a significant bend, the shear stress due to flow on the outside of the bend would be increased accordingly by the multiplier bend factor, F_m. (Design Chart 2.15).

Table: Boundary Stress versus Resistance

	Boundary Shear Stress (N/m^2)	Shear Resistance (N/m^2)
Stream channel Bottom	96	0.63
Stream channel Sides	104	0.39

As shown in the table above, the boundary shear stress greatly exceeds the shear resistance of the natural material on the stream channel bed and banks. The in-situ material would readily be eroded. Therefore, protection will be required to prevent erosion of the stream channel banks and bed.

Bank Protection Design

It is proposed to line the stream channel with appropriate sized rock protection up to the level of the design high water level. If the proposed lining material has a significantly different Manning's roughness coefficient (n) than the existing natural material, the stage / discharge relationship will change accordingly. For that case, the hydraulic radius would change, affecting the results of this analysis. For the purposes of this example, it is assumed that Manning's n-value is unchanged.

Try rock with D_{50} = 300 mm and angle of repose, $\phi = 42^o$.

Stream channel Bottom

$$\tau_{cb} = 0.0642 * D_{50}\ (mm) \qquad (5.31)$$
$$= 0.0642 * 300.0\ mm$$
$$= 19.2\ kg/m^2 = 188\ N/m^2$$

Stream channel Sides

$$\tau_{cs} = K_{cs} * \tau_{cb} \qquad (5.36)$$

where:

$$K_{cs} = (1 - (\sin^2 \theta / \sin^2 \phi))^{0.5} \qquad (5.37)$$
$$K_{cs} = (1 - (\sin^2 26.5^o / \sin^2 42^o))^{0.5} = 0.75$$

$$\tau_{cs} = K_{cs} * \tau_{cb} = 0.75 * 19.2\ kg/m^2$$
$$= 14.4\ kg/m^2 = 141\ N/m^2$$

Table: Boundary Stress versus Resistance

	Boundary Shear Stress (N/m^2)	Shear Resistance (N/m^2)
Stream channel Bottom	96	188
Stream channel Sides	104	141

As shown in the table above, the shear resistance of the riprap protection exceeds the boundary shear stress exerted by the flow for both the stream channel bottom and sides. Due to the extreme erodibility (very low shear resistance) of the natural material, the rock protection should extend across the bottom of the creek and up the side slopes to the design high water level. The horizontal limits of the rip rap coverage should extend along the stream channel banks as required to protect the structure and road embankment.

Hydraulic Design of Fish Habitat Structures

For fish habitat to be successfully designed, it must meet the biological requirements of the fish and be able to sustain various hydraulic conditions of the stream. Consequently, when undertaking the design of fish habitat, the project will require team work between a fish biologist, a hydraulic engineer as well as professionals of allied disciplines, such as a vegetation specialist.

This section covers the hydraulic aspects of designing fish habitat. For comprehensive coverage of all aspects of fish habitat, reference is made to the MTO Environmental Manual Fisheries. The only fish habitat design features covered here are those that may significantly affect river flows and, hence, require consideration of hydraulics.

Hydraulic Design Considerations for Specific Fish Habitat Structures

For preliminary design aspects, refer to Chapter 3.

The following effects of any in-stream works should be considered in design:

- backwater effects, if the conveyance of the river is changed,
- flow velocities,
- fish passage during periods of low flow,
- avoid ponding of stagnant water during periods of low flow,
- durability of proposed works,
- possible increase in erosion of adjacent river bed and bank areas,
- ice effects, such as increased ice jamming,
- ability of in-stream works to withstand the forces of flow.

A partial list of fish habitat structure types is as follows:

- sills (ie. weirs);
- deflectors;
- rock structures; and
- cover.

Sills

Sills or weirs are low structures that function by extending across the waterway and are designed to create pools and riffles through the action of scour and material deposition. For instance:

- broad crested sills, constructed of logs, rocks, gabions, concrete, sheet metal or various combinations thereof, can be used to form riffle patterns; while
- sills, constructed with sloping crests, gaps or notches, can be used to maintain scour holes.

Sills are more effective at creating a pool-riffle pattern along low-gradient streams and small rivers. Some design guidance is provided below.

- Maximum height; approximately 1/3 bank full water depth or 0.5 m, whichever is less.
- Location; straight reaches, keyed in to stream bed and banks.
- Minimum key trench depth; approximately twice the sill height above the bed.
- Stabilize; stream bed below sill.
- Shape; slope sills away from banks. Sill low point may be determined by considering minimum pool depth, maximum pool length and elevation of upstream rapids.
- Spacing; sills constructed too high or too close together may not create riffles.

The design must consider the impact of sills and weirs on the formation of frazil ice and downstream ice accumulation during winter months. Caution should be undertaken as sills can create low dams that could form a barrier to fish migration. Discussion on hydraulic theory related to weirs is presented in Chapter 8.

Deflectors

Deflectors are habitat structures that protrude from one bank and do not extend across the entire channel width. The primary use of deflectors is to create scour pools by constricting the channel and accelerating flow. Scoured material forms riffle areas, clean gravels and habitat for macro-invertebrates.

The following guidelines can be applied to the design of deflectors.

- Location; downstream from riffle areas.
- Maximum height; approximately 0.3 m above early summer baseflow water level but not high enough to influence backwater.
- Layout; angled upstream, downstream or perpendicular to the bank and to a maximum of 1/3 the channel width, should be keyed into bed and banks.
- Shape; often trapezoidal in shape and slope toward the channel. Opposing paired deflectors called double wing.
- Protection; stream banks opposite deflectors may require protection. This can be achieved by examining potential scour, tractive force or shear stresses and selecting a suitable lining. Refer to sections on Scour, Stream Channel Lining Materials and Stream Channel Erosion Analysis Methods.

Wing deflectors will concentrate the flow to create pools and should be constructed upstream of an erosion area. The deflectors should alternate between stream banks to encourage a natural meander pattern. Some general guidelines for installing wing deflectors are as follows.

- Avoid installation in unstable flood plains or braided channel reaches.
- Riprap/rock and/or gabions should not be less than 600 mm in diameter.
- Gabions should only be used when natural materials are not available.
- Deflectors should form a 45^{o} with the stream bank and must be well protected.
- Minimize scour of the wing deflector foundations appropriately.
- Rock revetments should be used with wing or current deflectors. Revetments are simple bank protection measures placed on the outside of meander bends and can be used as cover for fish. The rock should be placed protruding into the current and contain numerous openings.

Rocks

Rocks are usually effective wherever flow velocities are greater than 1 m/s. Boulders can be used to provide cover with natural overhangs and undercut faces along the stream bed.

The following design guidelines can be used.

- Rocks should be sized to withstand design flow velocities. For design flow velocities up to 3 m/s use rock with a minimum diameter of 700 mm. For design flow velocities of 4 m/s use rock with a minimum diameter of 1300 mm. Maximum size of rock should be approximately equal to 1/5 the channel width with the rock only protruding slightly above the low flow water surface.
- Rocks should be placed in groups. In groups of 3 or 5, rocks should be placed in straight reaches (riffles) or in meanders. Spacing, with the long dimension placed normal to flow, should be 2 or 3 m apart.
- Rocks should be spaced approximately one large rock per 30 m^2 of channel plan area.
- Igneous and metamorphic rocks are preferable to sedimentary rocks.
- The stability of the rock clusters may be checked using tractive force principles. (Design Example 5.9).

Channel Habitat Cover

Habitat cover provides fish with shade, shelter and concealment from predators and is usually designed to simulate bank side vegetation.

Habitat cover includes large boulders or gabions, floating rafts anchored to the bed or bank, ledges supported by pilings, trees and brush mattresses anchored to the bank and allowed to trail in the current and logs or haft logs aligned vertically with the flow and anchored to the bed with steel rods. Logs have a variable life span and are subjected to attack by ice, floating debris and decay. Soil bioengineering techniques may also be considered (Chapter 10).

Construction Considerations

Dewatering and Temporary Diversion

Dewatering and/or stream channel diversions to keep a site dry during construction can be a significant component of the cost of open stream channel works. The designer should consider the feasibility of working in naturally dry areas before undertaking diversions and dewatering.

The following factors should be considered when designing temporary construction diversions.

- Characteristics of stream channel flow, such as type of flood, season, magnitude and duration.
- Worker safety, duration of work, impact of flood damage and cost of delays.
- Requirements of fish
- Methods of diversion include:

    - installation of temporary metal, wood or other material flumes;
    - impounding of the stream channel flow followed by dewatering of the construction site.

- The permeability of stream channel subsurface materials.
- Specifications describing the contractors obligations.
- Cost.

The design should consider the need and feasibility of dewatering during construction. Water should be disposed so as not to be injurious to public health and safety. Pumped water can be discharged to sediment control ponds.

Dewatering should incorporate measures to minimize impacts on groundwater and surface water.

ln-stream channel construction, such as cofferdams, falsework and formwork may constrict or divert stream channel flows causing scour and bank erosion.

Excavation and Backfill

The use of inappropriate backfill can aggravate scour. Backfill in cohesive soils should be clay, coarse gravel or riprap. Granular stream channel bed foundation excavation may be backfilled with material similar to the natural bed. The Sediment and Erosion Control Plan should address environmental constraints, such as fish requirements.

Utilities

Contact should be made with utility companies early in the project. Moving the location and/or elevation of the utilities may be costly and involve significant project delays. Test pits can be dug to accurately locate underground utilities. This investigation should be carried out in cooperation with the utility agency concerned.

Changes in Design During Construction

The hydraulic engineer should be notified when any proposed construction alterations may affect the hydraulic performance of the works which may require approval of the hydraulic engineer. Any construction alterations should be documented on the *as-built* drawings.

Energy Dissipators

Energy dissipators are hydraulic devices used in channels to create or confine a hydraulic jump to dissipate energy in flowing water. If excess energy is not dissipated, it may create problems downstream, such as serious bank erosion or destructive turbulent flow. This energy is represented by the velocity head of the flow. The velocity head ($V^2/2g$) represents an equivalent vertical height, m, that is representative of the kinetic energy of the flow.

One of the most common types of energy dissipator is the drop structure. It may be a vertical or inclined drop. A drop can be provided in an open channel or in a pipe. Another type of energy dissipator is the stilling basin. It is used less often and is intended for dissipating high energy where an ordinary drop structure may not be adequate. These types of works are very disruptive to the natural environment and therefore should only be considered for situations where less disruptive means are not adequate. Drop structures may not be permitted in fish bearing waterways because they may present a barrier to migration.

The principles of hydraulic jumps are discussed in Chapter 8. Application of these principles in the design of energy dissipators is discussed in this section.

Vertical Drops

Vertical drop structures are usually constructed of riprap, gabion baskets or armour stone. The drop height is typically 1.0 m or less because of the limiting shear resistance of the lining materials. Figure 5.37 shows a typical vertical drop structure.

A vertical drop will create a hydraulic jump at the foot of the drop. According to *Open Channel Hydraulics*, French, (1985), the hydraulic relationships of a vertical drop are:

$$D = q^2/(g*h^3) \tag{5.38}$$
$$L_d/h = 4.30\ D^{0.27} \tag{5.39}$$
$$d_p/h = 1.00\ D^{0.22} \tag{5.40}$$
$$d_1/h = 0.54\ D^{0.425} \tag{5.41}$$
$$d_2/h = 1.66\ D^{0.27} \tag{5.42}$$

where:

- D = drop number (dimensionless)
- q = Discharge per unit crest width, m^3/s
- h = Height of drop (refer to Figure 5.36), m
- L_d = Distance to toe of jump from drop wall (refer to Figure 5.36), m

d_1 = Depth at beginning of jump (refer to Figure 5.36), m
d_2 = Sequent depth to d_1 (refer to Figure 5.36), m
d_p = Depth of pool under nappe (refer to Figure 5.36), m

Figure 5.36: Hydraulic Jump at the Foot of a Drop

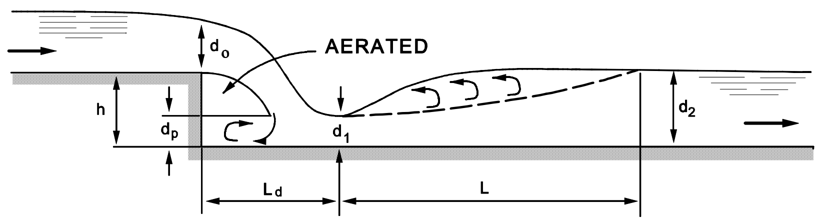
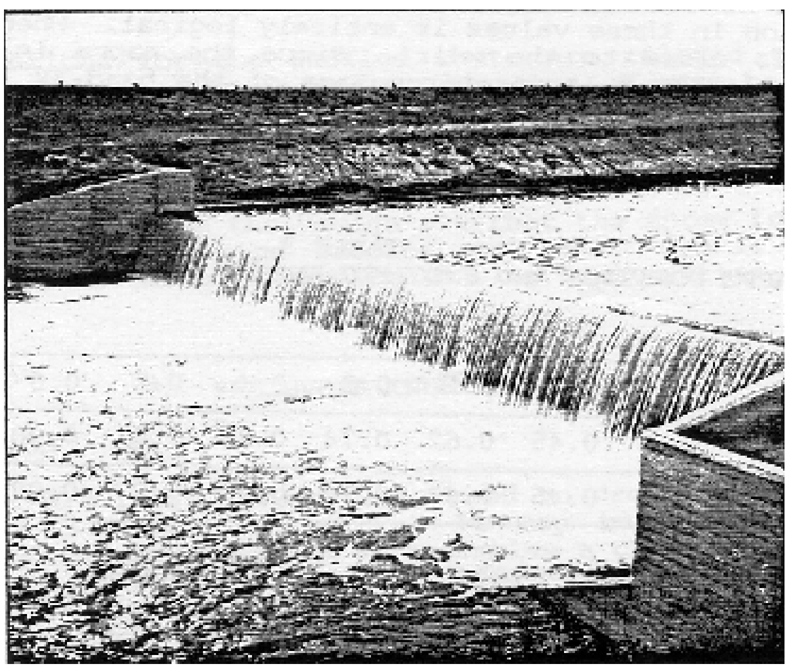


Figure 5.37: Typical Vertical Drop Structure (C.D Smith, 1985)

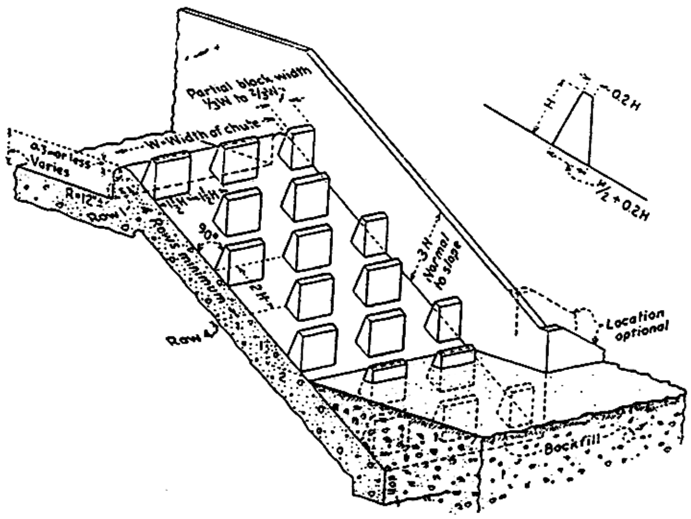


Chutes

Chutes are inclined drops and may be used as an alternative to a vertical drop. They are particularly useful for creating a series of drops to reduce a large difference in elevation between the two ends of a channel. A baffled chute is illustrated in Figure 5.38.

The information required to determine design sizes and basin dimensions of a chute is summarized in Design Chart 2.39.

Figure 5.38: A Typical Baffled Chute (C.D.Smith, 1985)



Stilling Basins

The most popular types of stilling basins are those developed by the U.S. Bureau of Reclamation (USBR). See Figure 5.39.The Froude number of the flow after a drop and before the jump is used to select an appropriate type of basin.

Froude Number, F_r	Basin Type
< 2.5	Basin I
2.5 - 4.5	Basin IV
>4.5 and velocity <15.2 m/s	Basin III
>4.5 and velocity >15.2 m/s	Basin II

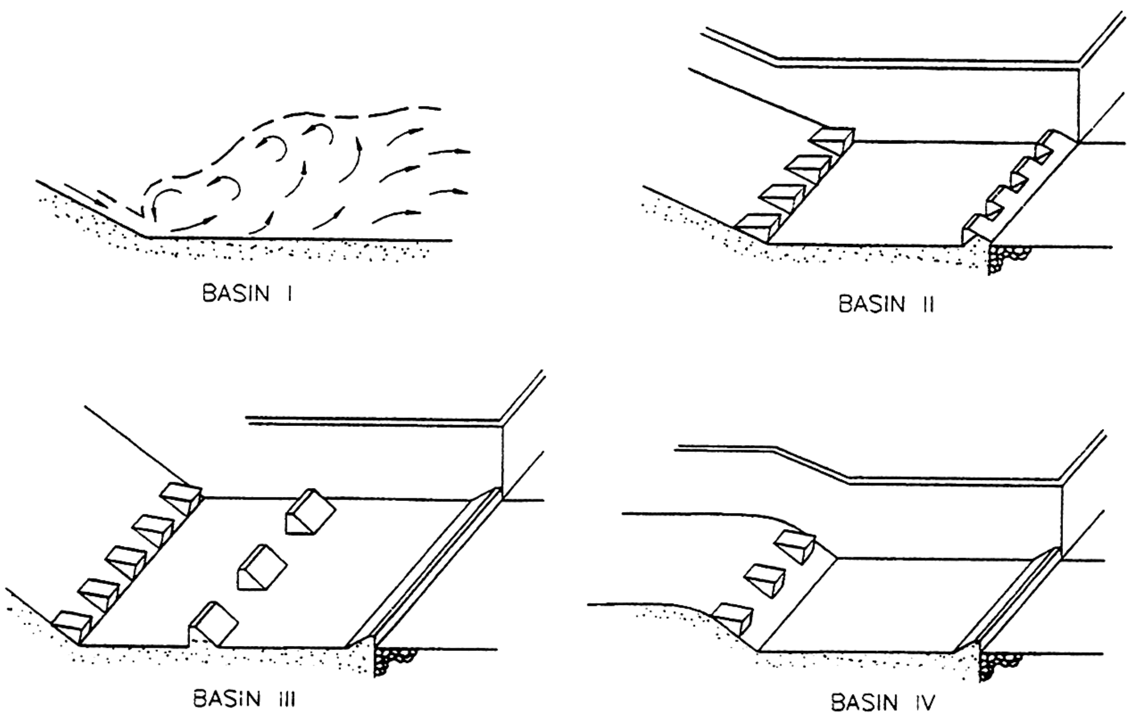
The following are characteristic dimensions of a stilling basin. They can be determined using Design Charts 2.39 to 2.41.

- Basin width.

- Basin length.
- Dimensions for chute blocks.
- Dimensions for dentated sill.

The floor elevation and the height of the side walls of the basin must be designed to ensure that the tailwater at the downstream end of the basin will not spill over the basin to adjacent areas. The tailwater must be of sufficient depth, such that the hydraulic jump is contained within the basin, otherwise the jump may tend to migrate downstream.

Figure 5.39: USBR Energy Dissipators



Design Example 5.10
Vertical Channel Drop

Required

Design a vertical drop in a channel. This drop is intended to replace the continuous steep invert gradient to reduce the velocity of flow.

Given

Existing Channel

- Bottom width, w = 3.0 m
- Cross section: trapezoidal with 2:1 side slopes
- Manning's n = 0.030 (Design Chart 2.01)
- Longitudinal slope, S = 0.0012
- Minimum depth = 1.0 m from the top-of-bank

Design Criteria

- Q_d = 3.0 m^3/s
- Froude number, F_r <= 4
- Allowable tailwater depth = 1.0 m for a flow of 3.0 m^3/s.

Proposed Drop Structure

- Width, w = 3.0 m
- Vertical side walls

Analysis

q = Q / w

where:

Q = Design flow, m^3/s
w = Width, m
q = discharge intensity = 3.0 m^3/s / 3.0 m = 1.0 m^2/s

Determine by trial the maximum change in elevation that can be accommodated by this drop for the given discharge intensity.

Calculate the drop number, D

$$D = q^2/(g * h^3) \tag{5.38}$$

where:

Acceleration due to gravity, $g = 9.81\ m/s^2$
Height of drop, $h = 1.0\ m$

$D = 1.0^2/(9.81 * 1.0^3) = 0.1019$

The following four equations incorporate D to proportion the spillway dimensions:

$$L_d/h = 4.30 * D^{0.27} \tag{5.39}$$

$$L_d = 4.30 * D^{0.27} * h$$
$$= 4.30 * 0.1019^{0.27} * 1.0 = 2.32\ m$$

$$d_p/h = 1.00 * D^{0.22} \tag{5.40}$$

$$d_p = 1.00 * D^{0.22} * h$$
$$= 1.00 * 0.1019^{0.22} * 1.0 = 0.605\ m$$

$$d_1/h = 0.54 * D^{0.425} \tag{5.41}$$

$$d_1 = 0.54 * D^{0.425} * h$$
$$= 0.54 * 0.1019^{0.425} * 1.0 = 0.205\ m$$

$$d_2/h = 1.66 * D^{0.27} \tag{5.42}$$

$$d_2 = 1.66 * D^{0.27} * h$$
$$= 1.66 * 0.1019^{0.27} * 1.0 = 0.896\ m$$

Calculate the Froude number, F_r, at the point d_1. This is where the depth of flow is a minimum and the flow velocity is a maximum.

$$F_r = V / (g * d)^{0.5} \tag{8.55}$$

where:

$V = Q / A$
$= q / (d_1 * 1.0\ m)$
$= 1.0\ m^3/s / (0.205\ m * 1.0\ m)$
$= 4.88\ m/s$
$g = 9.81\ m/s^2$ (acceleration due to gravity)
$d = d_1 = 0.205\ m$
$F_r = 4.88/(9.81 * 0.205)^{0.5}$
$= 3.44$

Repeat the above steps for other values of h. The results are shown in the table below.

Table: Flow Characteristics for a Range of Values of 'h'

$q = 1.0\ m^2/s$				
h (m)	D	d_1 (m)	V (m/s)	F_r
0.5	0.8155	0.247	4.04	2.60
1.0	0.1019	0.205	4.88	3.43
1.5	0.0302	0.183	5.46	4.07
2.0	0.0127	0.168	5.95	4.63

The table above shows that, for a drop of 1.5 m, the Froude number is 4.07 or approximately 4.0. Therefore, a value of h = 1.5 m or less is acceptable.

The dimensions of a vertical drop of height 1.5 m are shown below. Reiterating the four equations from above, with D = 0.0302:

L_d = 2.51 m
d_p = 0.694 m
d_1 = 0.183 m
d_2 = 0.968 m

The length of the hydraulic jump will vary, depending on the tailwater depth. If the tailwater depth is less than the sequent depth, d_2, the jump will extend downstream. If the tailwater depth is greater, the jump will be submerged.

Conclusion

For this case, a vertical drop of up to 1.5 m is acceptable. Greater drop heights may be designed where flow depths are greater, provided that the Froude number does not exceed 4.0.

Design Example 5.11
Incline Drop of USBR Type IV

Required

An incline drop is required to lower the elevation of a channel and reduce channel flow velocity. Consider using a USBR type drop structure.

Given

Design Criteria

- Width, w = 5.0 m
- Drop height - not limited
- Vertical walls, height = 2.0 m
- Chute slope, S = 0.05
- Manning's n = 0.015 (Design Chart 2.01)
- Design flow $Q_d = 10\ m^3/s$ (25 year return period)

Analysis

Calculate the normal depth of flow in the chute, using the Manning equation:

$$Q = \frac{A * R^{0.667} * S^{0.5}}{n} \qquad (8.66)$$

By trial and error:

Depth, $Y_1 = 0.314$ m
Velocity, V = 6.36 m/s

Calculate Froude number, F_r:

$$F_r = V / (g * d)^{0.5} \qquad (8.55)$$
$$= 6.36 / (9.81 * 0.314)^{0.5}$$
$$= 3.6$$

For a Froude number of 3.6, the USBR type IV stilling basin is recommended.

Design Example 5.12
Hydraulic Design of Channel Chute

Required

A rectangular diversion chute is to convey flow on a steep incline (10 percent gradient). Design the chute and stilling basin to develop the hydraulic jump and dissipate energy.

Given

Design Criteria

- Manning's n = 0.012 (Design Chart 2.01)
- Longitudinal slope of chute, S = 0.1
- Maximum depth of flow in chute = 1.5 m
- $Q_d = 20\ m^3/s$ (25-year return period)
- chute width, w = 3.0 m

Analysis

Applying the Manning equation:

$$Q = (1/n) * A * R^{0.667} * S^{0.5} \quad (8.66)$$

Assume that the site allows sufficient length and height of the chute to be provided for the flow to accelerate and reach a steady velocity.

By trial and error, estimate depth of flow, y
For depth of flow, y = 0.50 m

$$Q = (1/0.012) * 1.50 * 0.375^{0.667} * 0.1^{0.5}$$
$$= 20.6\ m^3/s$$

$$V = Q / A$$
$$= 20.6\ m^3/s / 1.50\ m^2$$
$$= 13.7\ m/s$$

Calculate Froude number:

$$F_r = V/(g * d)^{0.5} \quad (8.55)$$
$$= 13.7/(9.81 * 0.5)^{0.5}$$
$$= 6.2$$

The Froude number is greater than 1 and, therefore, the flow is supercritical. A hydraulic jump will occur where the flow transitions to subcritical. The ratio y_2/y_1 may be obtained as follows:

$$y_2 = 0.5[(1 + 8 * F_r^2)^{0.5} - 1] * y_1 \qquad (5.43)$$
$$= 0.5[(1 + 8 * 6.2^2)^{0.5} - 1] * 0.5 \text{ m}$$
$$= 4.14 \text{ m}$$

Calculate length of jump:

L/y_2 = 6.1 (Design Chart 2.39)
$L = L/y_2 * y_2$
= 6.1 * 4.14 m
= 25 m

The length of the hydraulic jump, L, is 25 m and the sequent depth, y_2, is 4.14 m. The tailwater level must be at or above the sequent depth or the hydraulic jump may migrate downstream. USBR stilling basin type III is recommended because the Froude number, F_r, exceeds 4.5 and the maximum flow velocity is less than 15.2 m/s. Determine the characteristic dimensions of the USBR stilling basin type III components: (Design Chart 2.40)

Initial depth, y_1 = 0.5 m
Sequent depth, y_2 = 4.14 m
H_{cb} = 1.7 * y_1 = 0.85 m; where: F_r = 6.2
H_{ds} = 1.3 * y_1 = 0.65 m; where: F_r = 6.2
L = 2.5 * y_2 = 10.4 m; where: F_r = 6.2

First teeth:

Width	= y_1	= 0.5 m
Spacing	= y_1	= 0.5 m
Height	= y_1	= 0.5 m
Side spacing	= 0.5 * y_1	= 0.25 m

Second teeth:

Distance from first teeth	= 0.8 * y_2	= 3.3 m
Width	= 0.75 * H_{cb}	= 0.64 m
Spacing	= 0.75 * H_{cb}	= 0.64 m
Side spacing	= 0.375 * H_{cb}	= 0.32 m
Top length	= 0.2 * H_{cb}	= 0.17 m
Back slope	= 1:1	

Sill:

distance from first teeth	= L	= 10.4 m
Front slope	= 2:1	
Height	= H_{ds}	= 0.65 m

Lake Crossings

Highway crossings over lakes may be at one or more of the following lake forms:

- main body of lake;
- lake narrows;
- bay or inlet;
- mainland to island or between islands;
- river inlet or outlet.

Crossing configurations may be categorized as follows:

- causeway through the lake with or without culvert or bridge opening. Complete closure of a gap is usually not recommended because it may create stagnant water or disrupt fish resources and navigation. Infilling of the lake should be minimized as it may disrupt fisheries and extensive fisheries compensation may be required.
- bridge span(s) providing a waterway opening.
- causeway and bridge combination.

Lake crossings present special problems which a designer must address. These include the following:

- seasonal and long-term variations in lake levels.
- possible bi-directional flows through a structure waterway opening.
- water level fluctuations resulting from changes due to wind setup, wave action, barometric pressure variance and artificial control.
- effects of causeway/bridge constriction on lake levels, discharges, flow velocities and scour.
- ice forces - vertical and horizontal - static and dynamic
- effects on fishery

A typical example of a lake crossing is shown in Figure 5.40.

The choice of a lake crossing should normally be determined at the preliminary design stage. (Reference: Chapter 3). Briefly, the choice depends on site conditions and design criteria. The most common solution for a highway crossing is to provide a causeway with one or more waterway openings. This may be necessary to satisfy hydraulic, environmental and navigation requirements.

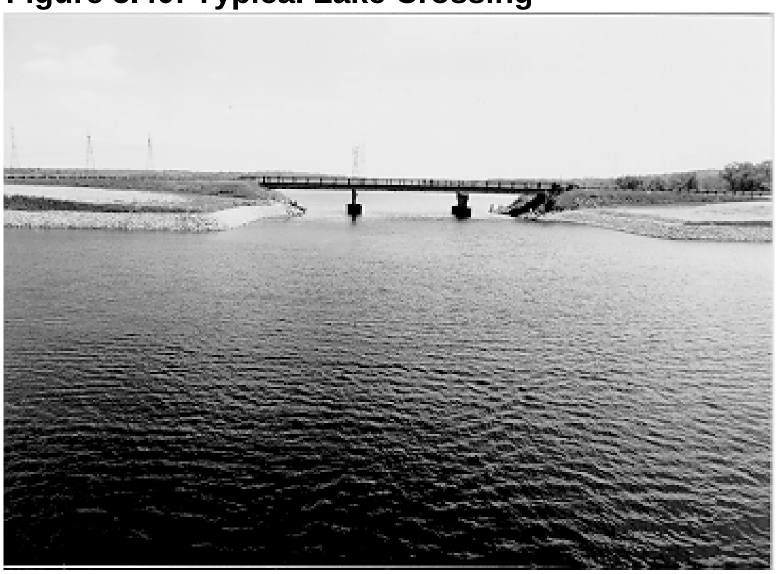
Lake Levels

Water levels in lakes are subject to seasonal hydrologic changes due to inflow and outflow, precipitation and evaporation. Short term variations are due to wind setup, seiches (water level variations due to wind and barometric pressure) may exceed 1.0 m in larger lakes. Water level changes resulting from hydro-electric power generation activities may be particularly large. Large and rapid changes in lake level may create large flows through a waterway opening, depending on the area of lake separated by the water crossing.

Estimation of water levels and changes using meteorological variables is a complex task. Therefore, high lake level estimates are generally based on field observations and records onto which wind and wave setup are added. Water surface elevations in a lake may vary seasonally and from year to year. In the Great Lakes, water levels are relatively low in winter and are higher in spring to fall, with fall levels generally the highest.

Records of water levels for major lakes in Ontario are available from Environment Canada. In addition, the Ontario Ministry of Natural Resources has completed studies of Great Lakes water levels. In the absence of established water levels, it may be necessary to consult long-term local residents, local libraries and archives to determine historic high lake levels.

Figure 5.40: Typical Lake Crossing

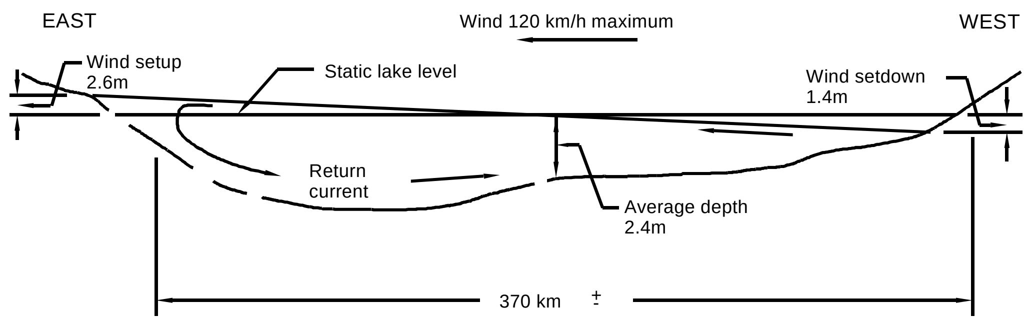


Wind Setup

Wind blowing over a body of water induces a surface current in the general direction of the wind. Horizontal current induced by wind is impeded in shallow water, thus causing the water level to rise downwind and fall at the upwind side. The term, *wind setup*, is used to indicate the rise in water level in downwind side, and as the fall in water level in windward direction is referred to as *setdown* (Figure

5.41). Factors affecting wind setup on lakes and inland bodies of water are fetch length, wind velocity, basin depth, wind duration, and basin geometry.

Figure 5.41: Wind Set-up and Set-Down



Fetch Length

Fetch length is defined as a length of water surface parallel to the wind direction. It is assumed that the wind occurs for sufficient duration to develop full wave activity, for the particular fetch length and wind speed.

Wind Velocity

Wind velocity data from meteorological stations can provide estimates for wind velocity over land. If winds are not measured at 10 m elevation, the wind speed must be adjusted accordingly. The following simple approximation is used to make this correction. This is applicable where distance above ground, z, is less than 20 m.

$$U(10) = U(z) * (10/z)^{1/7} \qquad (5.44)$$

where:

U(10)	=	Wind velocity measured 10 m above ground, m/s
U(z)	=	Wind velocity measured at height, z, above ground, m/s
z	=	Height above ground, m

Due to topography, the influence of the wind velocity over water is generally greater than over land. A wind speed correction factor is applied to account for reduced obstruction to wind over water as a function of fetch length (Design Chart 5.63). Design wind speed for a recurrence interval of 50 to 100 years is frequently selected. The wind speed used in calculations is the average sustained wind speed, and not the maximum recorded for short-duration gusts. Typical design wind speeds range from 80 to 160 km/h.

Determine the following characteristic dimensions of the USBR type IV stilling basin:

(Design Chart 2.41)

- Chute block dimensions
  - Maximum tooth width = Y_1 = 0.31 m
  - Tooth height above base = 2 * Y_1 = 0.62 m
  - Minimum tooth length = 2 * Y_1 = 0.62
- Dentated sill
  - Height = Y_1 = 0.31 m
- Minimum tail water depth, Y_2 = 4.6 * Y_1 = 1.43 m (where F_r = 3.6)
- Minimum basin length, L = 5.6 * Y_2 = 8.0 m (where Y_2 = 1.43 m)

Basin Depth

The basin depth is usually taken as the average depth below the water surface over the fetch length.

Duration

The length of time that wind blows in the same general direction over a wave generating area or fetch.

Wind Setup Calculation

The equation for wind setup based on the Netherlands Zuider Zee formula is as follows:

$$S_u = (f * u^2) * \cos\theta / (63000*d_f) \tag{5.45}$$

where:

S_u = Wind setup height, m
u = Wind speed, km/h
f = Fetch length, km
d_f = Average depth over fetch length, m
θ = Angle between fetch length and wind direction.

The actual setup may be somewhat more or less than calculated and should be adjusted since the above equation applies to a rectangular reservoir of constant depth.

For irregular basins, an additional multiplier factor, A, calculated as follows, may be included in the equation,

$$A = 2(2*b_o + b) / 3(b_o + b) \tag{5.46}$$

where:

b_o = width of windward shore, km
b = width of leeward shore, km

Wave Effects

The effect of waves should be considered when designing bridges, causeways, embankments or other works in lakes. Wave height, run-up, and force are the main factors to consider.

Wind generated waves are not regular. The pattern of waves on a body of water exposed to wind generally contains waves of many periods. Typical records from a gauge during periods of high waves indicate that heights and periods of real waves are not constant as is assumed in theory. Wave length and direction of propagation are also variable. For detailed discussion of wave and water level predictions, see reference: U.S. Corps of Engineers publication, *Shore Protection Manual*.

Wave prediction techniques require the use of complex computer models and meteorological data. However, simplified techniques based on field measurements and records, can be used to predict approximate but acceptable wave heights for the design of highway facilities along inland lakes.

Significant Wave Height and Period

A given wave train contains individual waves of varying heights and periods. In coastal hydraulics, a term *significant wave height*, H_s, is defined as the average height of the highest one-third of all the waves in a wave train. This is expressed as $H_{1/3}$. The significant wave height is the design wave height normally used for designing flexible revetments for inland lakes and waterways.

Predicting Wind Generated Waves

The height of wind generated waves is a product of fetch length, wind speed, wind duration, and depth of water. Simplified wave prediction relationships may be used to establish probable wave conditions for the design of highway embankments in lakes and large inland bodies of water. The following assumptions are implicitly built in these simplified methods.

- The fetch is short with a length of 120 km or less.
- The wind is uniform and constant over the fetch.

Water depth affects wave generation for a given set of wind and fetch conditions. Wave heights will be smaller and wave periods shorter if wave generation takes place in shallow water rather than in deep water. The height of a wave generated is also influenced by the fetch and duration of sustained wind speed.

Rational formulations for forecasting wind generated waves for relatively shallow water are not available. The method in Design Charts 5.24 to 5.33 based on U.S. Army Corps of Engineers, *Shore Protection Manual*, applies deep water forecasting relationships to shallow lakes by determining wave energy due to wind stress and adjusting for energy lost due to bottom friction and percolation for shallow conditions.

The required direction, duration, and velocity of wind can be determined from meteorological data, and the fetch length and depth determined from plans and photographs. As an alternative, shallow water waves may be determined using the method of Thijsse and Schijf (Design Chart 5.34). For the prediction of deep water waves, the nomograph (Design Chart 5.35) may be used.

Wave Action on Structures

A structure may be subjected to non-breaking, breaking and broken waves during different water level stages. Forces due to non-breaking waves are primarily hydrostatic. Breaking waves exert an additional force due to dynamic effects and compression of entrapped air pockets. Dynamic forces may be much greater than hydrostatic forces and structures located where waves break are designed for greater forces than those exposed only to non-breaking waves. Non-breaking waves may occur when the depth of water in front of a structure is greater than 1.5 times the maximum expected wave height.

When designing for a breaking wave condition against a structure, it may be necessary to determine the maximum breaker height to which the structure will be subjected to. The design breaker height, H_b, depends on the depth of water at a distance seaward from the structure toe where a wave first begins to break.

If the maximum design depth at a structure toe, the incident wave period, and the slope in front of structure toe are known, design breaker height can be obtained using Design Chart 5.36.

Wave Run-up

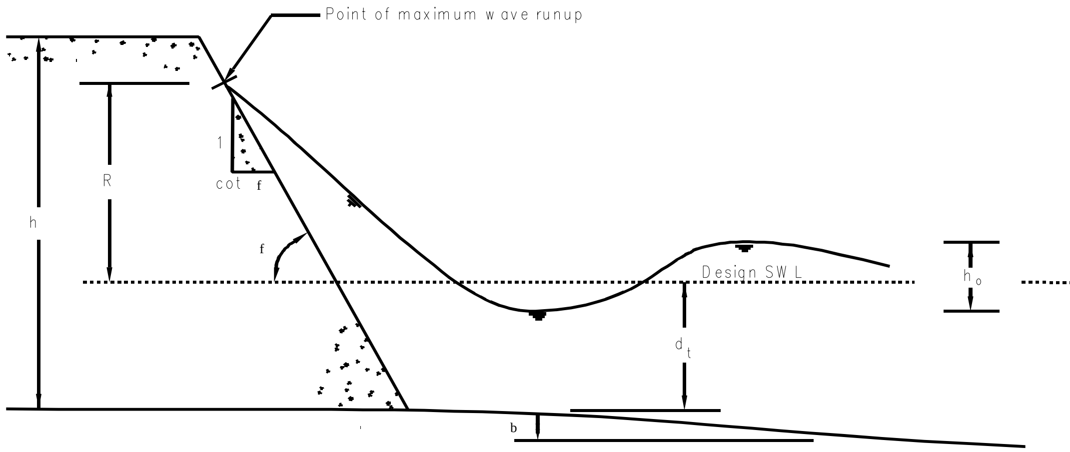
The vertical height above still water level to which water from an incident wave will travel up a structure slope is termed *wave run-up*. Figure 5.42 shows a schematic definition of wave run-up.

An estimate of wave run-up, in addition to wave height, is required to establish the top elevation of highway slope protection. Run-up is a function of structure slope, roughness, water depth at structure toe, bottom slope in front of structure and wave height and period. Using Design Chart 5.37, wave run-up can be estimated for smooth impermeable slopes when $d_t/h_o \geq 3.0$. The wave run-up so obtained should be corrected for model scale effects using Design Chart 5.38. In order to apply Design Chart 5.37 (prepared for roughness of impermeable concrete surfaces) for slopes of other roughness, a roughness and correction factor, r, described below is applied (Design Chart 5.21).

$$r = \gamma \text{ (rough slope)} / \gamma \text{ (smooth slope)} \qquad (5.47)$$

The detailed procedure for composite slopes and for d_t/h_o values less than 3, is available in U.S. Army Corps of Engineers, *Shore Protection Manual*.

Figure 5.42: Typical Wave Run-up



where:

R = Run-up height, m
h_o = Unrefracted deep water wave height, m
d_t = Depth of water at structure toe, m
ϕ = Slope of structure
ß = Bottom slope in front of structure

Slope Protection Design

Where wave action is predominant, a protective lining composed of layers of random-shaped stones, together with a layer of rock or armour stone may provide an effective measure. For design wave heights less than 1.5 m, angular graded riprap is generally the most suitable protection; for waves higher than 1.5 m, it is more economical to use uniform-size armour units.

Design of Cover Layer

The following formula based on model testing results of Iribarren (1938) and Hudson (1953, 1959), may be applied to determine the stability of armour rock on rubble structures:

$$\omega = \omega_r * H_s^3 / K_d(S_r-1)^3 * \cot \phi \qquad (5.48)$$

where:

ω = Weight of an single armour unit in the primary cover layer, N
ω_r = Unit weight of armour unit, N/m^3
H_s = Design wave height at structure toe, m
S_r = Specific gravity of armour unit relative to water, $S_r = \omega_r/\omega_w$
ω_w = Unit weight of water, 9810 N/m^3
ϕ = Angle of side slope
K_d = Stability coefficient that varies with shape of armour unit, roughness of unit surface, sharpness of edges and degree of interlocking. (Design Chart 5.22)

Cover layer slopes steeper than 1.5:1 are not recommended. Equation 5.48 relates the weight of an armour stone of nearly uniform size to other properties. For a graded riprap armour stone, the relationship is modified to:

$$\omega_{50} = \omega_r * H_s^3 / (K_{rr}(S_r\text{-}1)^3) * \cot \phi \qquad (5.49)$$

The symbols are the same as defined in Equation 5.48. The maximum weight of graded rock is usually taken as 4.0 * ω_{50} with a minimum value of 0.125 * ω_{50}). K_{rr} is a stability coefficient for graded riprap similar to $K_{d.}$ Values of K_{rr} are also given in Design Chart 5.22. The use of graded riprap cover layers is more applicable to revetments where the design wave height is less than 1.5 m. For higher waves, uniform size armour stone estimated by Equation 5.48 should be used.

Selection of Stability Coefficient

Stability coefficient K_{rr} (Design Chart 5.22) combines the effects of all the variables other than wave height, specific gravity of water and structure slope, including:

- Shape of armour units
- Number of stones in cover layer
- Method of placement of units, random or individually placed
- Surface roughness and sharpness of edges of armour units
- Type of wave defence structure
- Part of structure (trunk or head), and
- Angle of incidence of wave attack.

Bottom Elevation of Primary Cover Layer

The armour stone in a cover layer should be placed down the slope to an elevation below the lake stage so that it is revetted with the bottom.

Toe Berm for Cover Layer Stability

Revetments exposed to breaking waves should be supported from below with a quarry stone toe berm. Quarry stones in the berm should weigh $\omega/10$ where ω is the weight, N, of a cover layer armour unit. The width of the top of this berm is estimated using the following expression for n=3:

$$B = n * K \ (\omega/\omega_r)^{1/3} \qquad (5.50)$$

where:

- B = Crest width, m
- n = Number of stones,
- K = Layer coefficient,
- ω = Weight of armour stone in cover layer, N
- ω_r = Unit weight of armour stone, N/m^3.

The minimum width of the berm is calculated using Equation 5.50 for n = 2, where B is equal to width of the berm.

Underlayers

The first underlayer directly beneath the cover layer should have a minimum thickness of two quarry stones (n = 2). These should weigh about one-tenth the weight ($\omega/10$) of the cover layer assuming that the cover layer and first underlayer are both quarry stone, or where the cover layer is concrete armour with a stability coefficient, $K_d \leq 12$.

Filter Layer or Bedding Blanket

Wave action against a rubble structure creates turbulence that erodes the underlying soil through voids of the structure. Larger amounts of rubble may be required to allow for loss due to undermining and settlement. Excessive settlement resulting from leaching, undermining and scour can be prevented by the use of either a filter blanket or bedding layer.

When the structure is founded on sand or fine gravel, a filter blanket should be provided to prevent differential wave pressures, current and ground water flow from removing sand and gravel through the voids of rubble. The graded filter requirements underlying a cover layer are the same as for an ordinary channel lining. Generally a geotextile, coarse gravel or crushed stone filter may be placed directly over sand but silty and clayey soils and some fine sands must be covered by a coarser sand first.

Design Example 5.13
Crossing over An Inland Lake

Required

Design a new bridge crossing and causeway across the narrows of an inland lake in order to carry a proposed highway.

Given

- Design Criteria

  The proposed structure and its location have been determined from an earlier preliminary design study and shown in the figure below.

  - The bridge opening to provide flood-free passage for inflow for storm events up to 100-year return period and provide protection against high lake levels and waves.
  - A minimum clearance of 1.0 m is required between the design high water level and the soffit elevation of the bridge. A freeboard of 1.0 m between the design high water level and the shoulder of the road embankment at low sag is also required.
  - Navigation requirement: a minimum vertical clearance of 3.5 m is required between average summer high water level of 177.7 m and the bridge soffit elevation, and a minimum horizontal clearance of 25 m between piers is needed.
  - There should be no adverse flooding to the upstream lands.

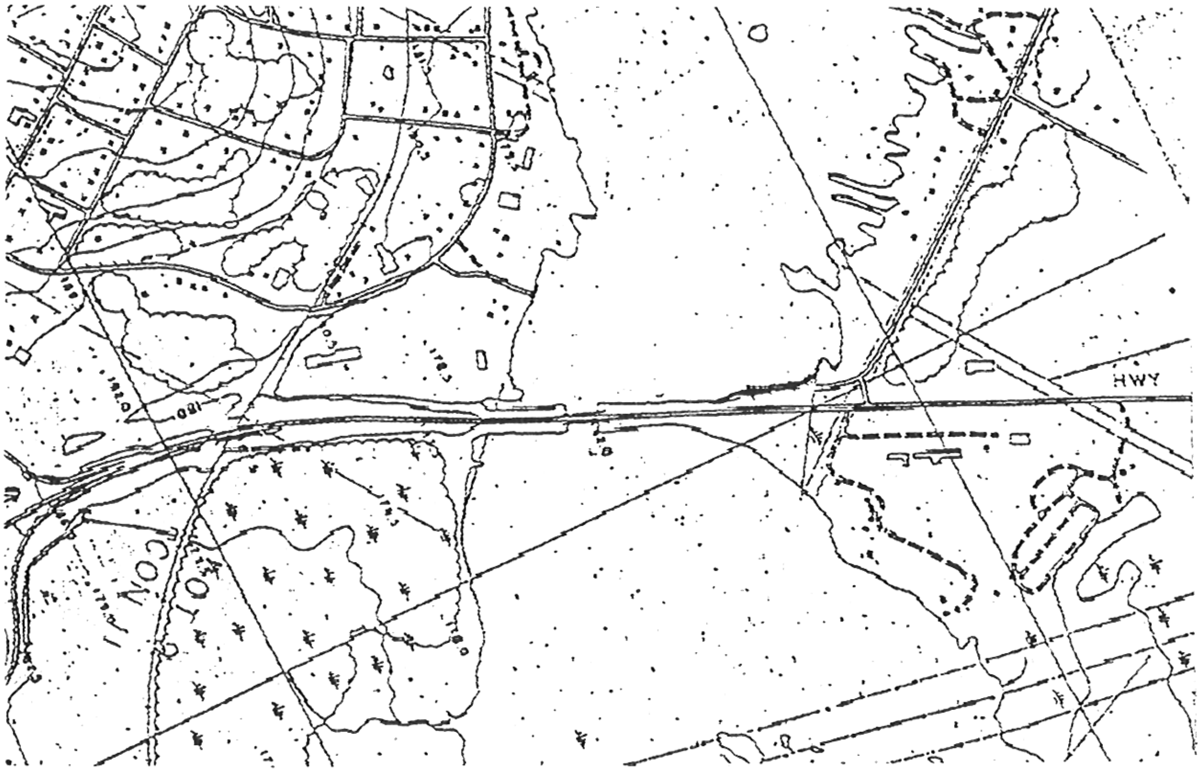
- Site Data

  - The inland lake covers an area of approximately 8 km^2 with an average depth below the autumn stage of 6 m. The width of the narrows of the lake at the proposed crossing site is approximately 450 m. The distance from the head of the lake (at the confluence with the stream) to the crossing site is approximately 3.0 km.
  - The soils investigations reveal bedrock at approximately 9 m below normal lake level with approximately 3.0 m of silty clay, sand and gravel overlying the bedrock.
  - Design wind speed = 120 km/h = 33.3 m/s

- Watershed Data

  The watershed, the lake and the proposed bridge crossing are shown in the figures below. The location of the proposed bridge crossing is downstream of the confluence of two streams, which flow into the inland lake. This, in turn, discharges into another small lake. The two rivers, which are 24 km and 37 km long respectively, combine to provide drainage for a total catchment area of 523 km^2.

Figure: Site Plan and Sections



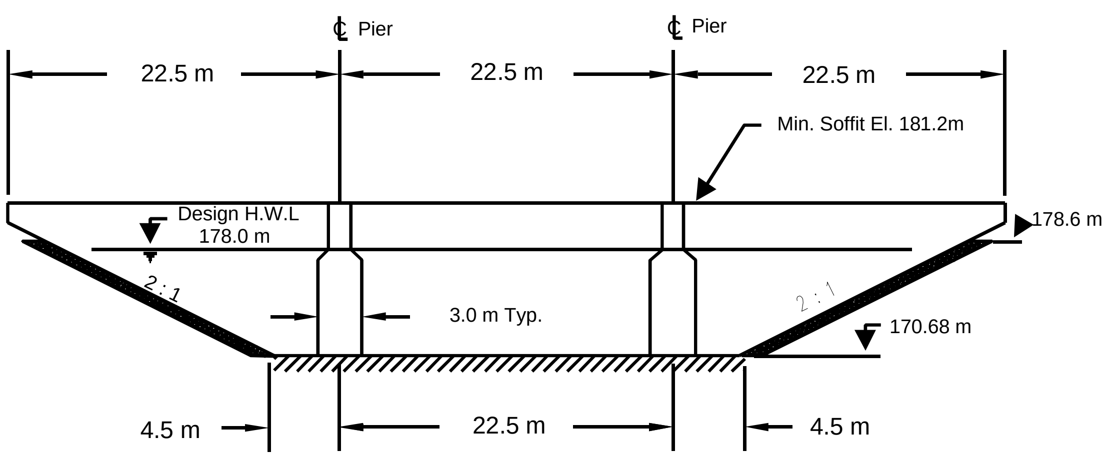
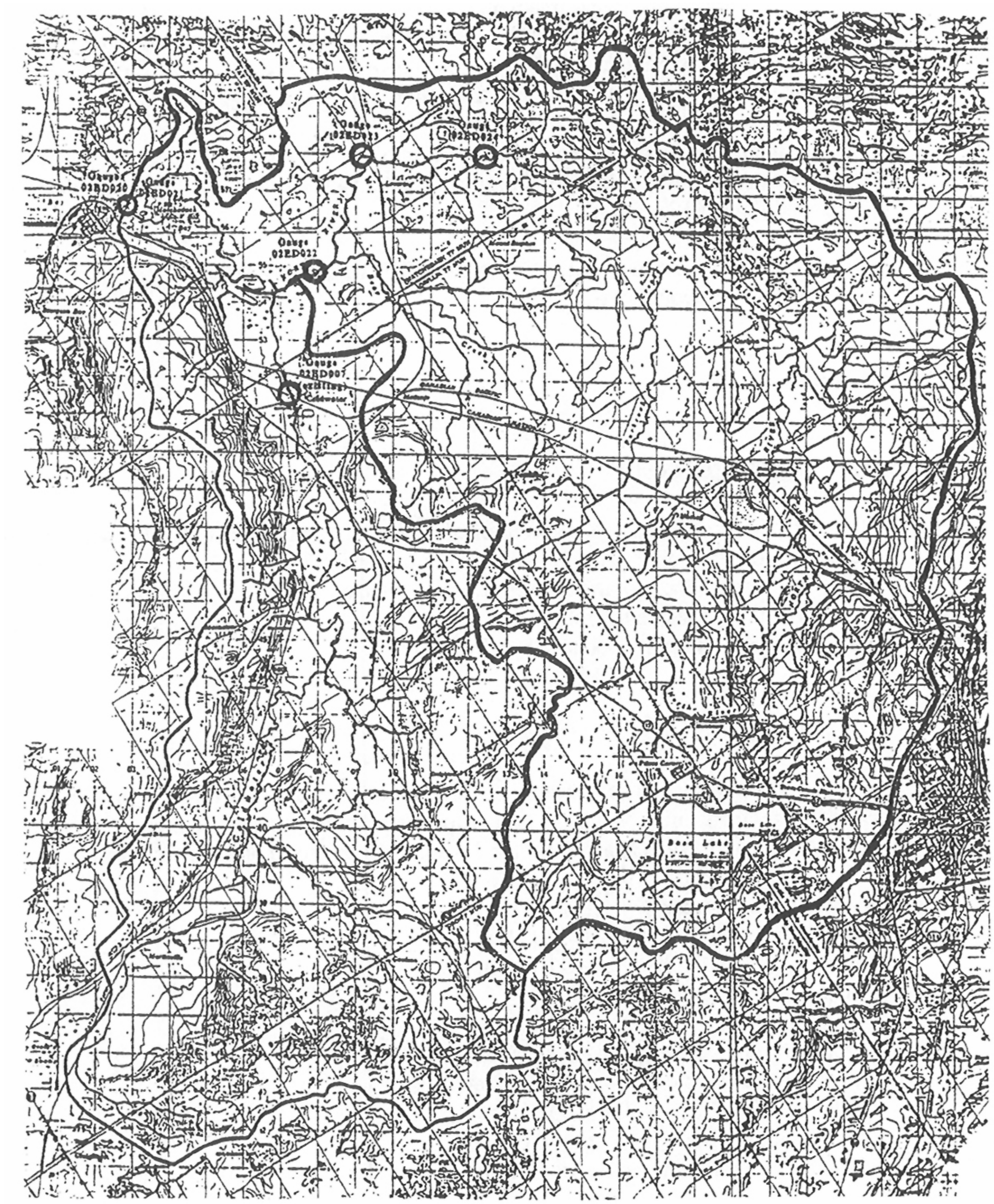


Figure: Watershed Plan



Flow Estimates

The proposed bridge site is downstream of a Water Surveys of Canada gauge located on one of the streams. Using the CFA 2.1 (Consolidated Frequency Analysis) Computer Model of Environment Canada (Chapter 8), a frequency analysis of the known data was carried out. The 100 year return period peak instantaneous flow was found to be 56.6 m^3/s at the gauge site.

The flow estimate was transposed using the area ratio method (Chapter 8) from the gauge site (drainage area ≈ 177 km^2) to the crossing site (total drainage area ≈ 523 km^2) to obtain the 100-year return period flow of 127.5 m^3/s.

This is a conservative estimate because storage of flow in the lake upstream of the site would tend to reduce peak instantaneous flows.

Hydraulic Analysis

Design High Water Level

Outflow from the downstream lake is controlled by a dam. The high water level downstream of the lake crossing corresponding to the 100-year return period is 178.00 m. This estimate, derived from recorded water levels does not include the effects of wind and wave setup. This estimate will be used as the design tailwater elevation for the hydraulic analysis to determine the design of the waterway opening including upstream flood levels and velocities.

Water Crossing

Try various waterway openings to determine an acceptable head water elevation and flow velocity. The HEC2 computer model may be used to proportion the waterway opening (reference: Design Example 5.3). The results of the final analysis for the 100-year design storm of 127.5 m^3/s are:

Span of waterway opening	= 62.5 m
Upstream bed elevation	= 170.68 m
Downstream bed elevation	= 170.68 m
Skew angle	= 0^o
Bridge width	= 25 m
Tail water elevation	= 178.00 m
Upstream flood elevation	= 178.018 m
Flow velocity	= 0.43 m/s
Required soffit elevation	= 179.018 m

Based on the design flow velocity, provide rip rap protection of the fill slope with D_{50} = 300 mm

Wind Setup within the Lake

$S_u \quad = (f * u^2) \cos\theta / 63000 * d_f$ (5.45)

where:

Fetch length, f	= 3.0 km
Design wind speed, u	= 120 km/h
Average fetch depth, d_f	= 6.0 m
Angle between fetch and wind direction, θ	= 0^o
$\cos\theta$	= 1
Maximum Wind Setup, S_u	= 0.11m

Seiche due to Barometric Pressure Change

Barometric pressure changes due to storms will have a negligible effect on waterlevels in small lakes, such as this. However, for a large lake, differences in atmospheric pressure over its area may cause a significant seiche. It is possible to have seiches exceeding 1.0 m on the Great Lakes.

Waves Generated within the Lake

The significant wave height and period are required for the determination of wave run-up and the designing of protective works. Determine the significant wave height and period, using Design Charts 5.24 to 5.33, given:

Fetch length, f	= 3.0 km
Average depth, d_f	= 6.0 m
Design wind speed, u	= 120 km/h
Duration	= unlimited

The wind stress factor is computed by the following equation.

$U_A \quad = \quad 0.71 * U^{1.23}$ (5.51)

where: Design wind speed, U = 33.3 m/s (u = 120 km/h)

U_A = 53.0 m/s

Significant wave height, H_s = 1.3 m, where depth is 6.0 m (Design Chart 5.27)
Significant wave period, H_t = 3.2 s, where depth is 6.0 m. (Design Chart 5.27)

Design of Embankment Protection

Cover Layer Design

For design wave heights less than 1.5 m, angular graded riprap is recommended. For a cover layer of graded riprap armour stone, determine W_{50}, the weight of stone exceeded by 50 percent of the layer sizes.

$$\omega_{50} = \omega_r * H_s^3 / K_{rr} (S_r-1)^3 * \cot \phi \qquad (5.49)$$

where:

Design wave height, H_s	= 1.3 m	
Unit weight of rock, ω_r	= 2640 kg/m^3	
Stability coefficient, K_{rr}	= 2.2	(Design Chart 5.22)
Specific gravity of armour, S_r	= 2.65	
Angle of structure slope, cotan ϕ	= 2	

The computation shows that:

ω_{50} = 293 kg
$D_{50} = 1.15 * (\omega_{50}/\omega_r)^{1/3}$ (5.52)
D_{50} = 0.55 m
= 550 mm

Maximum weight of graded rock, $\omega_{max} = 4 * \omega_{50}$ = 1172 kg
Minimum weight of graded rock, $\omega_{min} = 0.125 \times \omega_{50}$ = 37 kg
Maximum diameter, D_{max} = 877 mm
Minimum diameter, D_{min} = 275 mm

Cover Layer Thickness

Calculate the thickness, r, of a layer of riprap.

$$r = n * K\Delta * (\omega_{50}/\omega_r)^{1/3} \qquad (5.53)$$

where:

Number of stones in layer, n	= 2	
Layer coefficient, $K\Delta$	= 1	(Design Chart 5.23)
Layer thickness, r	= 0.96 m	

Underlayer Design

The stones in the underlayer should weigh about one-tenth the weight ($\omega/10$) of those in the cover layer and have a layer thickness of n = 2.

Repeat the procedure for the cover layer thickness to obtain the maximum and minimum stone sizes and layer thickness for the underlayer.

Wave Run-up

The vertical height, γ, above the normal water level to which waves will travel up an embankment was calculated in order to determine the elevation of the roadway embankment. In this example it is assumed that the bottom contours are parallel and aligned in the same direction as wave crests. Therefore, the refraction coefficient is equal to 1 and $h_o = H_o$.

Input Data:

h_o = 1.3 m
T = 3.2 s
d_t = 6.0 m
cotan α = 2.0

Computation:

d_t/h_o = 6.0 m /1.3 m
= 4.6

The estimated value of (d_t/h_o) is greater than 3.0. (Design Chart 5.36)

$h_o / (g * T^2) = 1.3 / (9.81 * 3.2^2)$
= 0.0129

γ/h_o = 1.7
γ = 1.7 x 1.3 = 2.21 m

Model scale correction coefficient, k = 1.15, based on model tests (Design Chart 5.38)

γ_k = 2.21 x 1.15
= 2.54 m

Design Charts 5.37 and 5.38 are based on run-up for smooth, impermeable slopes. A roughness and porosity factor, r, obtained from Design Chart 5.21 is used in the following equation to account for run-up on riprap covered slopes.

r	= γ (rough slope)/ γ (smooth slope)	(5.47)
r	= 0.60, for angular rock riprap (i.e. quarry stone)	(Design Chart 5.21)
0.60	= γ (rough slope)/2.54 m	
γ (rough slope)	= 0.60 * 2.54 m	
	= 1.52 m for a riprap covered slope	

Riprap protection should be placed up to a minimum of 1.52 m above the design high water elevation of 178.018 m to elevation 179.53 m.

Estimating General and Local Scour

Methods of estimating scour given in this chapter can be used to calculate maximum general and local scour (Design Example 5.6).

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Appendix 5A: Data Requirements

Generally, the scope and amount of data required for a project varies between projects. The more complex a project is, or the more potential for drainage issues to arise from a project, the greater the data requirements will be.

Water Crossings and Channels

Site Plan

A plan of the site typically includes the topography, proposed road centreline, property boundaries, fence lines, utilities and existing structure(s), etc. Other details shown may be relative to the type of proposed work.

Stream Channel Alignment

In addition to the information available from the site plan, useful information may be provided by aerial photographs. The stream channel alignment in a plan view may help to determine the skew of the crossing in relation to the existing stream alignment. This is especially critical for fish bearing streams and streams prone to ice and debris flows or scour.

Channel Cross Sections

Channel cross sections, preferably plotted to a natural scale (same horizontal and vertical), and their locations are useful for hydraulic assessments and design of a water crossing. The cross sections should preferably include all breaks in the natural ground.

Stream Invert Profile

A surveyed profile of a stream invert is required to estimate the local slope of the stream. This profile should extend along the stream, both upstream and downstream of the crossing site, preferably taken along the deepest part of the channel. The length of the coverage along will depend on site conditions, prevailing channel slope and the complexity of the project. Including water level elevations at the time of survey for the invert profile will be desirable.

Invert Profile along Channel Realignment

The invert profile along the proposed realignment will be required to design the invert elevations along the channel realignment.

Stream Channel Roughness

Where a typical stream surface roughness is expected, a site visit is advisable to gain knowledge of the site for selecting appropriate values of Manning's roughness coefficient, n (Design Chart 2.01). Where a flood stage and discharge are known, Manning's n may be calibrated by using known channel characteristics, with Manning's equation or a water surface model such as HEC-2 or HEC-RAS.

High Water Marks

High water marks from past floods are useful for determining an appropriate hydraulic design of a waterway opening.

If the investigation is carried out soon after a high event, high water marks may be established by observing debris deposited over banks or caught in bushes and trees. The water line prevailing at the time of flood, may also be detected by contrast in the appearance of vegetation. The vegetation submerged in water will appear different than grass and willows above a water line. Several high water marks should desirably be established to provide correlation of the collected information.

If it is desired to establish high water mark for a past flood of several years ago, interviewing local area residents, or review of site photographs, local archives, libraries, etc. may be required.

High water marks established should be tied-in to geodetic elevations with a field survey.

High Ice Marks

Historic high ice marks from significant ice runs provide a useful insight for designing a water crossing. Scars on tree trunks caused by flowing ice are permanent indicators. For longer term information, it is desirable to examine older trees on river banks. Ice abrasion scars may also be found on banks and structures. Ice scars on structures, may be observed on abutments and piers and girders.

It is also advisable to interview local residents to determine the history of ice problems, flowing ice elevation, size of ice floes and any resulting damage.

Ice marks established in the field could be translated to survey elevations with a field survey.

Channel Scour

Knowledge of the extent of channel scour at an existing crossing may be required to carry out any remedial measures. Scour measurements may also be required to estimate the lateral erosion or bed lowering at abutment or pier foundation.

Long-term lateral channel scour and adjustment may be determined from historical aerial photography. A localized movement may require periodic measurements.

Lake Crossings

Additional data required for lake crossings are as follows:

- inflow discharge at lake crossings,
- record of lake levels,
- water depths,
- wave height data
- wind data (speed, direction, and duration),
- fetch length.

Record of Water Levels

Records of water levels for large lakes in Ontario are available from Environment Canada. Alternatively, historical high lake levels may be established by interviewing long-term local residents and by consulting local archives, libraries, etc.

Lake Depths

Sounding may be required where sufficient information on lake depths is not available.

Wave Data

A single storm is usually represented by a significant wave height. Records of past storms may be used to plot a frequency of occurrence curve from which the significant wave height for a given return period may be determined.

Wind Data

Historical records of wind characteristics, i.e. speed, direction and duration, are usually obtained from meteorological stations in the form of hourly wind speeds, and direction. The period of record required is usually a minimum of one year. If direct records are not available, wind speeds may be estimated from analysis of air pressure charts. Estimates by this method should be used with caution.

Fetch Length

In lakes, fetches are limited by surrounding land forms. The fetch taken in the direction of prevailing wind may be estimated from hydrographic maps.

Appendix 5B: Typical Bridges, Culverts and Transition Structures

This section is an introductory discussion on hydraulic aspects of typical bridges, culverts and their general design arrangements. A few stream channel forms are also shown as an illustration of the settings of streams in which bridges and culverts are typically built.

Bridges

A bridge may have a single or multiple spans. Bridge waterway openings may vary in shape from rectangular with high abutment walls to trapezoidal with spillthrough abutments and fills. "Spillthrough" is a term meaning that the abutment is formed by filling part of the watercourse. Bridge piers may be supported on shallow footings, piles or caissons. A bridge may be constructed on square (perpendicularly) or on skew (at an angle) to the river. Typical bridges and their general design arrangements are shown in Figures 5.43 to 5.44.

Figure 5.43: Bridge Opening - Elevation View

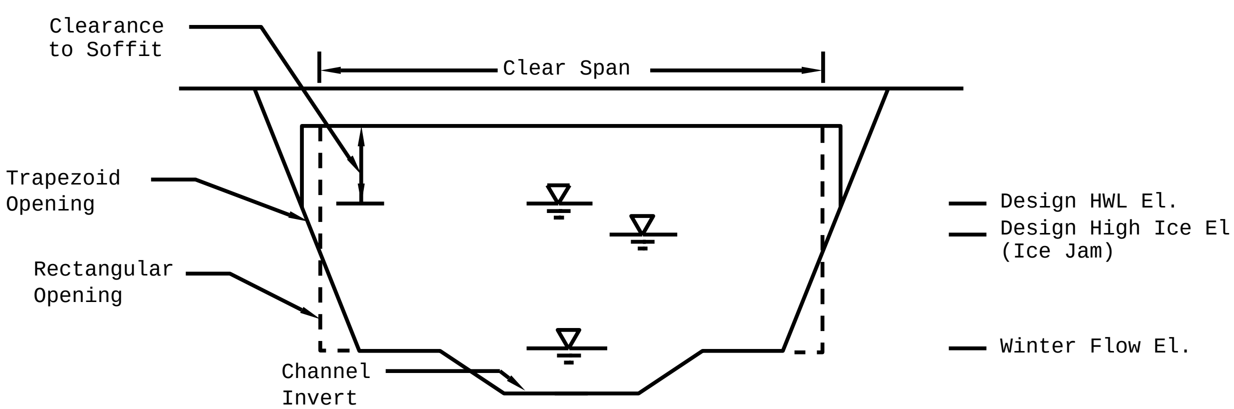
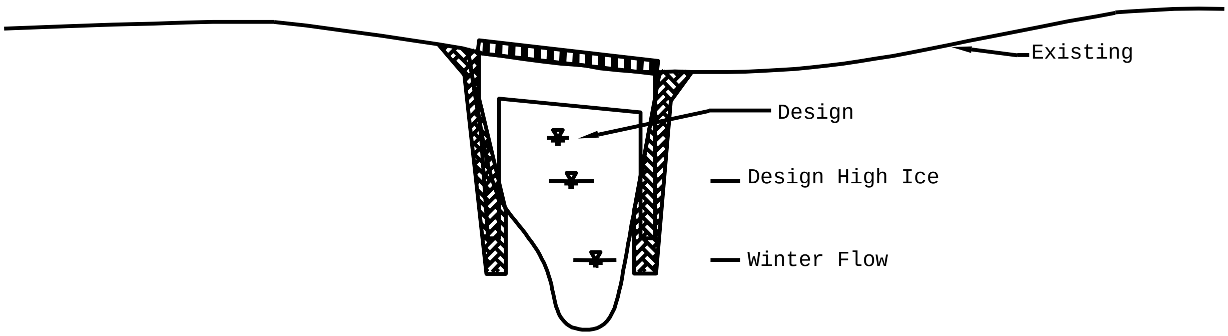


Figure 5.44: Bridge Opening - Road Profile



Culverts

A culvert may have a single or multiple barrels. The barrel cross section shape may be circular, square, rectangular, arched, or elliptical. Most culverts are constructed as closed-invert structures, however, occasionally an open-invert may be used. Typically, open-invert culverts are based on a sound, stable and non-erodible foundation. Typical culverts and their general design arrangements are shown in Figures 5.45 to 5.48.

Figure 5.45: Culvert Crossing - Plan View

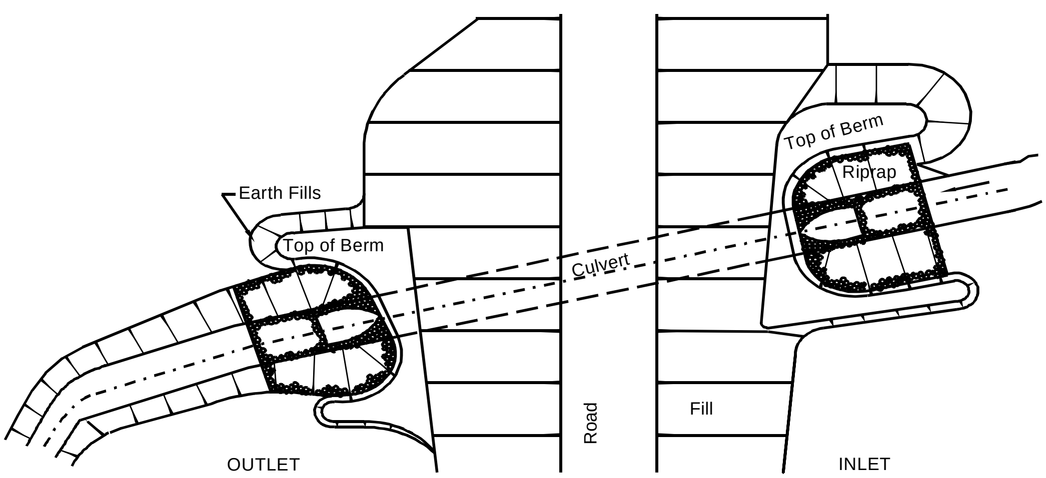


Figure 5.46: Culvert Crossing - Profile View

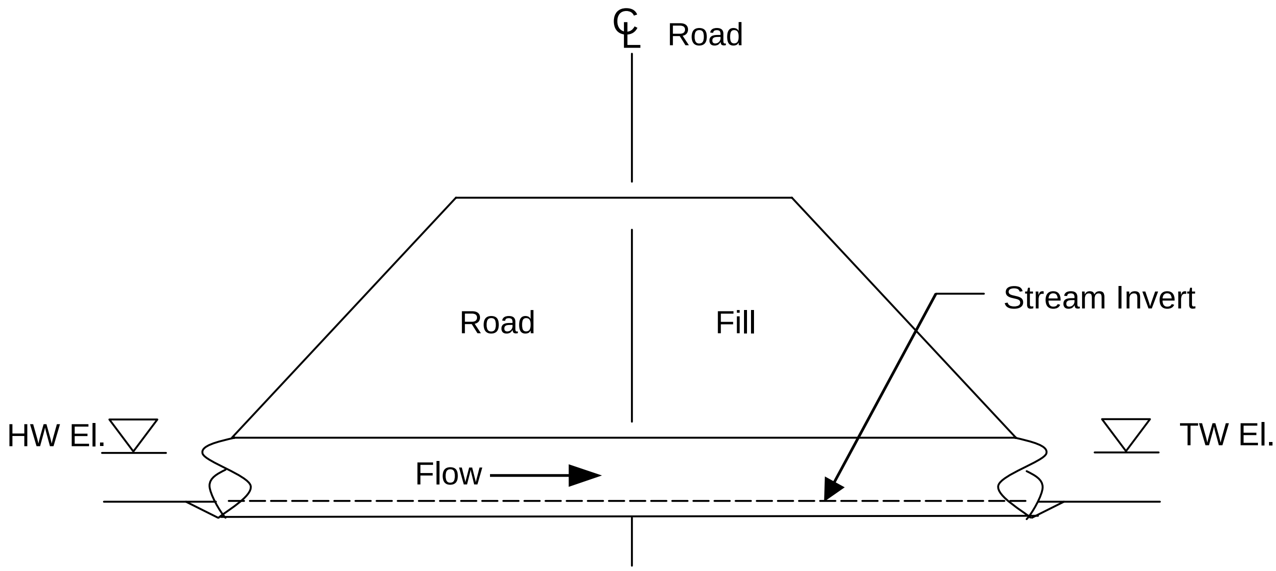


Figure 5.47: Culvert End Treatment - Steel Culverts

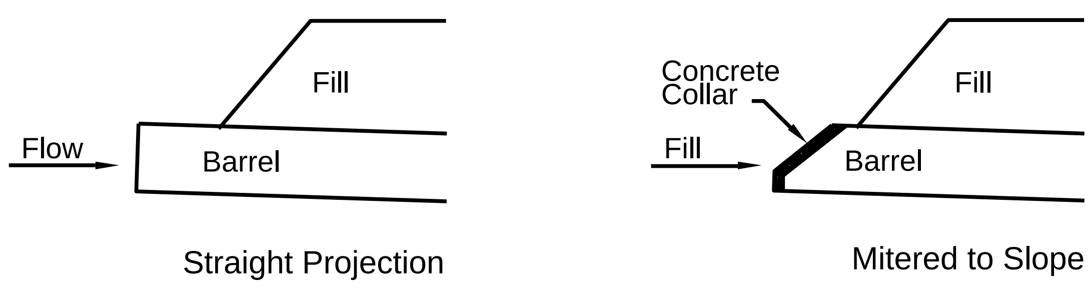
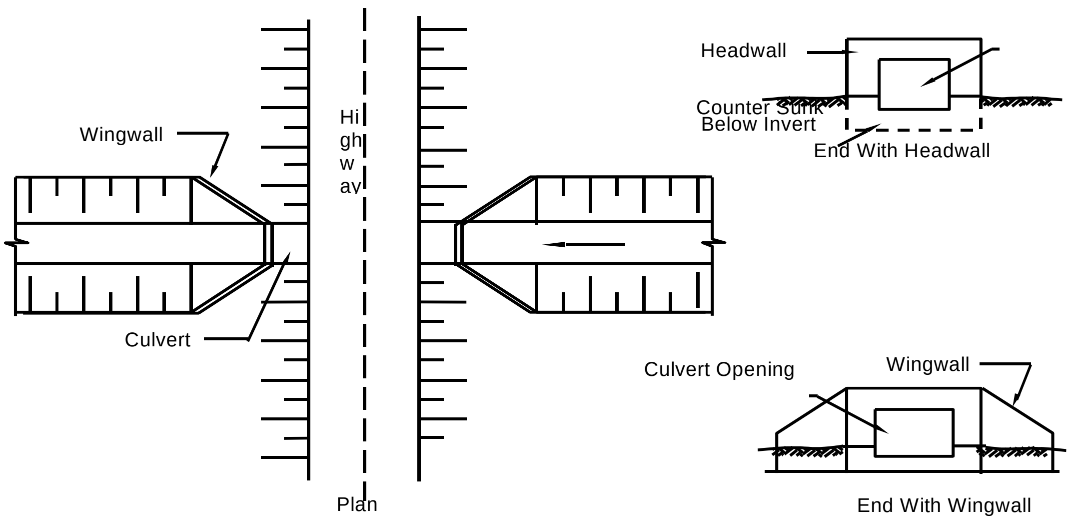


Figure 5.48: Culvert End Treatment - Concrete Culverts



Bridge/Culvert Transitions

Where a bridge (or culvert) creates a constriction to the flow of a river, it may be necessary to provide a transition zone to smooth the flow as it approaches the structure. Commonly used transitions are discussed below.

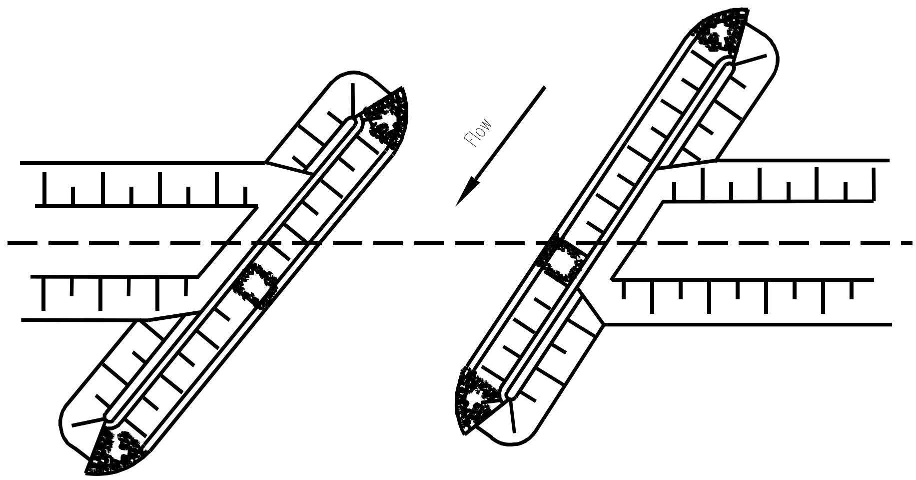
Approach and Bridge Fills

Approach and bridge fills are the most commonly used transition configuration. The extent of transition depends on the height and width of the crossing, the skew and its lateral position in relation to the river and flood plain.

Guide Banks

Guide banks are earth embankments constructed beyond standard bridge and approach fills. They are used in fairly difficult situations. They usually have an elliptical shape. Guide banks may be used to improve flow alignment and skew angle, protect susceptible areas from erosion, and deflect floating ice and debris. A typical guide bank is shown in Figure 5.49.

Figure 5.49: Typical Guide Bank on Skew



Revetments

Revetments are stream banks reinforced with riprap or other linings to guide the flow along a bank(s), which may otherwise be eroding.

Groynes (Spur Dikes)

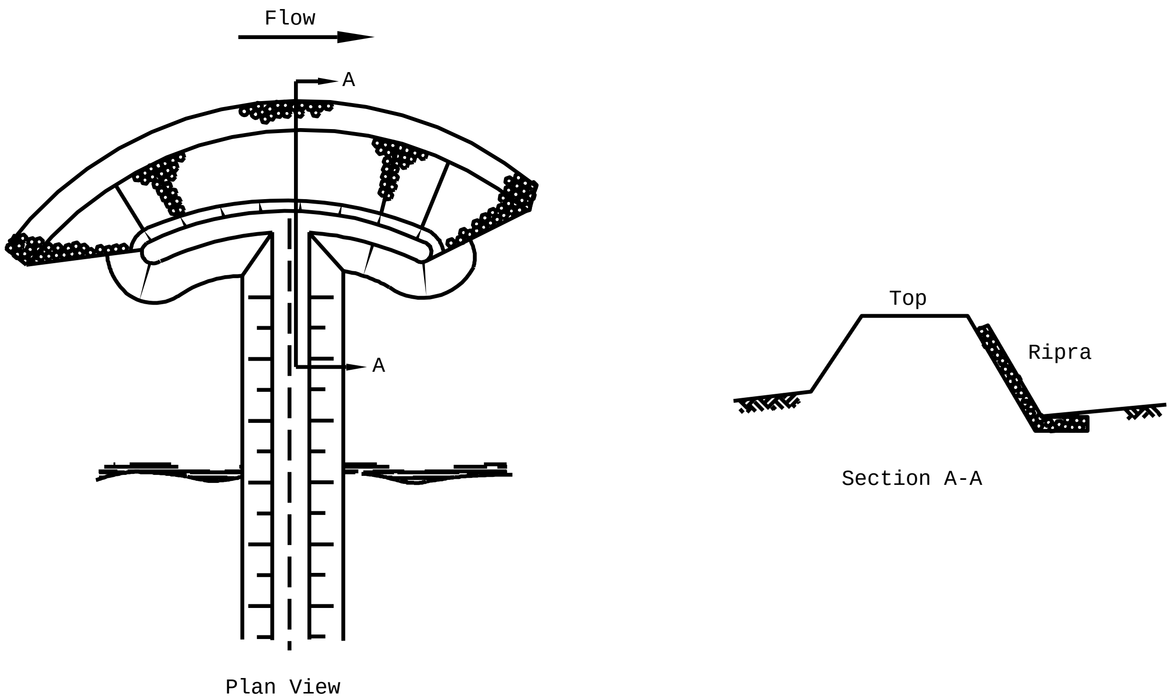
Groynes or spur dikes are finger-like structures usually constructed of earth fill, extending from a stream bank into the flow. Occasionally these may be constructed of steel sheet piles or of timber cribs (earth-filled inside). Figure 5.50 shows a typical earth-filled groyne.

Groynes, usually in combination with guide banks, are used to improve flow transition and alignment towards a waterway opening. They may also be used to deflect flows away from areas susceptible to bank erosion. As groynes obstruct flow, a scour hole may form nearby. The nose therefore usually requires armour protection. Low level groynes may be used to enhance fish habitat by creating pools.

Dikes

Dikes are usually used to contain large unpredictable flow on flood plains and away from the main channel. They are constructed of earth fill and are generally not provided with protective cover.

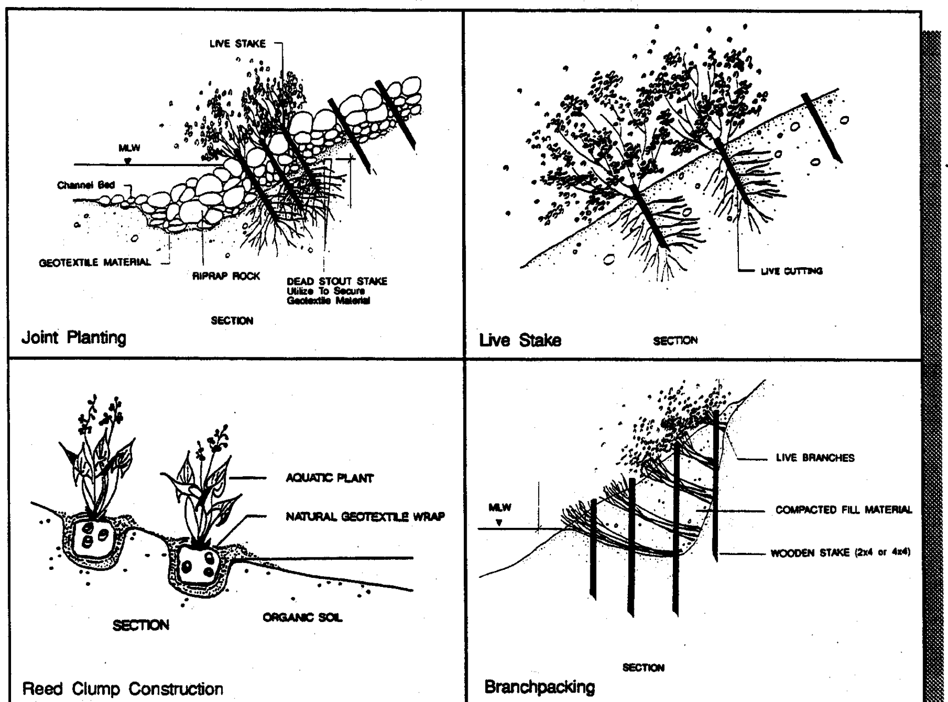
Figure 5.50: Typical Guide Bank / Tee-Head Groyne



Appendix 5C: Fact Sheets

Lined Channel

SOIL BIOENGINEERING



Source: After Robbin B. Sotir & Associates, 1991.

Section Shape

- trapezoidal
- triangular
- parabolic

Maintenance

- strengthening sites to repair damaged plants, remove alien plant species
- newly constructed sites inspected every year

Typical Characteristics

- methods include live fascine, brushmattress, brushlayer, live cribwall, branch packings, Lorenz drain, reed clump, joint planting, drain and live stake
- choice of method dependent on soils, slope, hydrophobia, fertility, acidity, erosion, growing season, rainfall, pests, traffic and maintenance

Computation Methods

- maximum permissible tractive force
- maximum permissible velocity

Cross References

- Bioengineering for Land Reclamation and Conservation, H. Schiechtl, 1980.
- Biotechnic Slope Protection and Erosion Control, D.H. Gray and A.T. Leiser, 1980.

Lined Channel (continued)

SOIL BIOENGINEERING

General Criteria/Guidelines

- application guidelines
  - install during dormant season
  - consider fish spawning and migration period
  - use local native wood cuttings
  - local within 80 km
  - harvesting - plants cut perpendicular to the stems - use chainsaws, weed eater saws, bush axes, etc.
  - protect cuttings during transporation
  - store in cool dry areas - heal cuttings in moist soil
  - install cuttings within 8 hours of harvesting
  - consult soils engineer to determine best soil mix for backfill and plant growth - chemical soil analysis required
  - construction - use small equipment, limit site access, locate staging areas, avoid critical time periods, avoid extensive earth works, retain natural vegetation, manage runoff, protect stockpiles, install erosion control measures

Drawing Requirements

- plan and profile
- design water levels on plan and profile
- lines and grades
- slopes
- revegetation locations and species
- stream and slope stabilization and materials
- in-stream works and materials
- design water levels on plan and profile

Specification Requirements

- supervision
- plant materials, size
- hauling distances
- soil materials composition and amount
- plant handling, storage and installation

Computation Steps

i) determine permissible shear stress
ii) estimate channel slope, side slope, bed slope, design discharge and estimate flow depth
iii) determine Manning roughness coefficient for estimated flow depth
iv) compute flow depth
v) compare computed flow with estimated flow depth.
    if greater than 0.03 m
    repeat steps (ii) through (iv)
vi) calculate the shear stress for the channel bed and side slopes.
    if the shear stress is greater than permissible
    repeat steps (i) through (vi)
vii) for channel bends:
    (a) determine shear stress factor in bends
    (b) calculate shear stress in bend
    (c) if shear stress is greater than permissible then a more erosion resistant lining is required
    (d) calculate superelevation
viii) compute water surface profile for flow rates greater than 1 m^3/s

Construction Steps

- preconstruction
  - -locate and secure harvesting sources
  - -fence off sensitive areas
- construction start up
  - -check earth works
  - -check live cuttings and earth material
  - -ensure site protection
- daily inspection during installation
- regular monitoring

Advantages

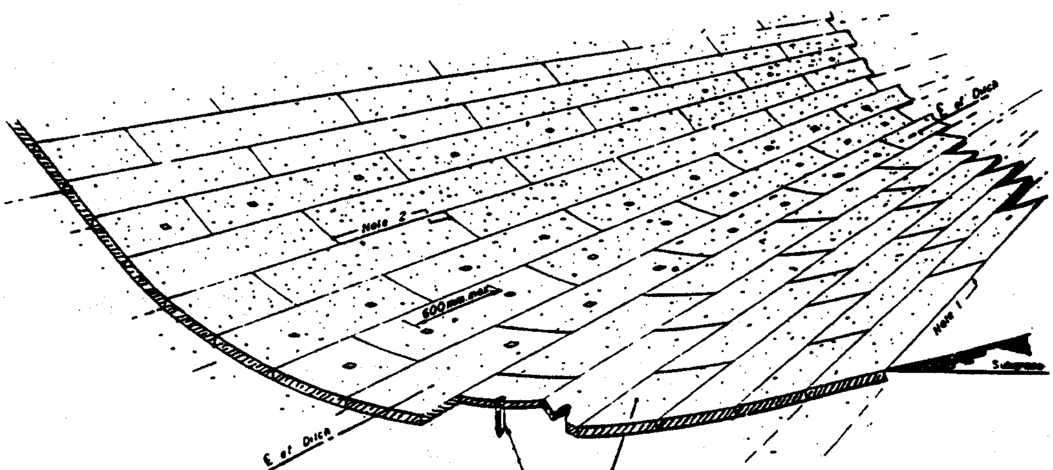
- wide range of environmental conditions
- natural appearance
- provides wildlife habitat
- enhances habitats
- flexible
- self supporting, self maintaining

Disadvantages

- vegetation vulnerable to trampling
- planting materials may not be local or abundant
- installation conducted during dormat season
- conventonal engineering may impose limitations

Lined Channel

VEGETAL



Slabs 19x19x290mm
Stakes to be driven flush but not deeper

Sod as per specifications.

NOTES:

1. Where topsoil and sodding is specified the top edge is to be recessed into the slope so as not to impede the flow of surface run-off.
2. Joints in adjacent rows shall be staggered.
3. On slopes of 2:1 to 3:1 - the bottom three rows and every third row shall be pegged. On slopes steeper than 2:1 each and every row shall be pegged.
4. All dimensions are in millimetres unless otherwise shown.

Source: OPSD 218.01 by Ontario Provincial Standards and Specifications, September 14, 1990.

Section Shape

- trapezoidal
- triangular
- parabolic

Maintenance

- immediate
  - fence
  - reseed/resod areas that do not take
  - cut to encourage growth
- long term
  - cut as required
  - regrade as required
  - fertilize and reseed (if necessary)

Typical Characteristics

- side slopes < 3:1 H:V
- minor ditches
- major channels
  - depth 3 to 10m
  - width 10 to 100m
- intermittent flow
- minor ditch slopes 0.5 to 5.0%
- major watercourse slopes 0.2 to 0.5%
- maximum flow velocity 1 to 2 m/s

Computation Methods

- maximum permissible velocity (Manning equation)
- maximum permissible tractive force
- for flows > 1 m^3/s perform backwater analysis

Cross References

- OPSD: 218.01, September 14, 1990.
- OPSS: 201, 501, 510, 514, 517, 570, 571, 572, 599, 902, various dates.

Lined Channel (continued)

VEGETAL

General Criteria/Guidelines

- cutting requires side slopes 3:1 to 4:1 (H:V) or flatter
- prevent undermining (seepage)
- provide toe protection above normal water level
- staggered joints for sodded channels
- banks not continuously submerged for more than a few days
- plant/seed in the late spring or early fall and avoid the spawning season
- grass mixtures dependant on site conditions
- typical Canada #1 lawn grass seed mixture includes: Creeping Red Fescue 55%, Canada Bluegrass 27%, Perennial Rye Grass 15%, and White Clover 3%
- specifications should include landscape preservation and water pollution prevention measures
- landscape preservation includes location and size of construction roads, contractors camp site, burrow areas, quarry sites and blasting precaution
- water pollution prevention includes erosion control measures, accidental spills, dewatering and aggregate processing wastewater

Drawing Requirements

- plan and profile
- design water levels on plan and profile
- application boundary and location
- stakes: size and spacing
- sod type
- slopes
- edge finishing
- design water levels on plan and profile
- areas at watercourse preservation

Specification Requirements

Sod

- dimensions
- pegging slopes
- alignment
- cutting/placing time
- payment
  - topsoil, m^3
  - sod, m^2
  - water, m^3

Seeding

- method
- time constraint
- seed mixture
- seed+mulch thickness
- coverage
- payment
  - topsoil, m^3
  - seed, kg
- ground preparation
- aftercare

Computation Steps

i) determine permissible shear stress
ii) estimate channel slope, side slope, bed slope, design discharge and estimate flow depth
iii) determine Manning roughness coefficient for estimated flow depth
iv) compute flow depth
v) compare computed flow with estimated flow depth.
   if greater than 0.03 m
   repeat steps (ii) through (iv)
vi) calculate the shear stress for the channel bed and side slopes.
   if the shear stress is greater than permissible
   repeat steps (i) through (vi)
vii) for channel bends:
   (a) determine shear stress factor in bends
   (b) calculate shear stress in bend
   (c) if shear stress is greater than permissible then a more erosion resistant lining is required
   (d) calculate superelevation
viii) compute water surface profile for flow rates greater than 1 m^3/s

Construction Steps

Typical Sod

- prepare surface (25mm)
- fertilize
- apply topsoil (50mm)
- place sod and compact
- hold sod in place with stakes
- water

Seed & Mulch

- prepare surface (25mm)
- place topsoil (50mm)
- seed and mulch
- apply fertilizer
- option - temporary matting

Advantages

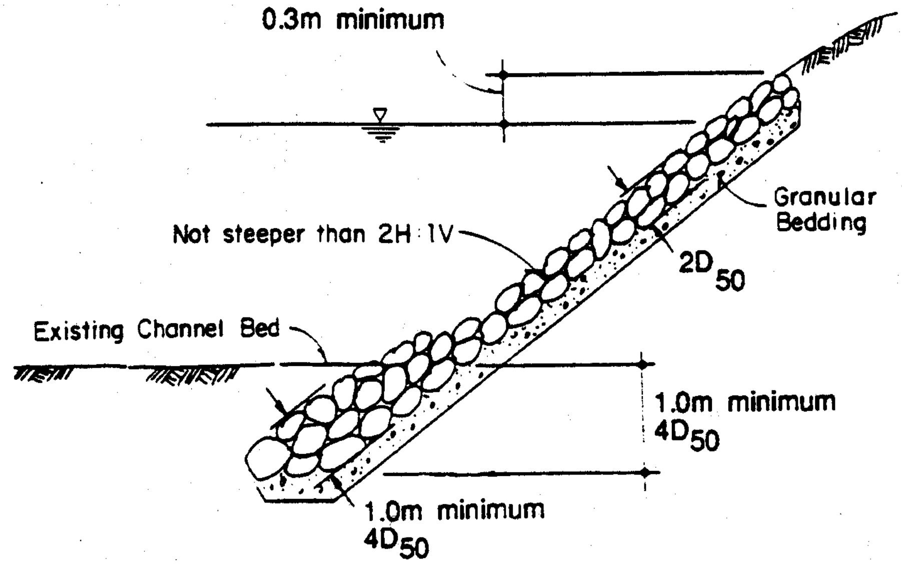
- improves aesthetics
- economical
- filters out sediment
- flexible
- low maintenance
- conventional equipment can be used

Disadvantages

- initial high maintenance costs
- sod slides on steep slopes
- new sod likely to be killed by drought
- will not withstand high flow velocities
- may have erosion problems until sod is established in the channel
- maximum flood duration approximately 2 days

RIPRAP

Lined Channel



TOE PROTECTION RIPRAP CHANNEL LINING

Source: Urban Storm Drainage Criteria Manual, 1968, by Wright-McLaughlin Engineers Engineering Consultants.

Section Shape

- trapezoidal
- triangular

Maintenance

- replace displaced stones
- removal of woody vegetation

Typical Characteristics

- side slopes flatter than 2:1 H:V
- flow velocities < 3.0 m/s
- Manning roughness coefficient for riprap is $0.0152\ D_{50}^{1/6}$
- critical tractive force for riprap is $0.0642\ D_{50}$
- riprap lining thickness $2.0 \times D_{50}$
- rock toe along the entire length of riprap
- granular or synthetic filter required
- undermining is a common cause of failure

Computation Methods

- maximum permissible tractive force

Cross References

- OPSS 201, 206, 501, 510, 511, 512, 514, 902, 1860, various dates
- Guidelines for the Design and Quality Control of Geotextiles, MTO
- A Handbook of Measurs to Protect Water Resources in Land Development, J.J. Tourbier and R. Westmacott, 1981.

RIPRAP

Lined Channel (continued)

General Criteria/Guidelines

- minimum granular filter thickness is 150 mm
- extra attention should be given to the design of the toe and terminals of the bank protection
- stone should be blocky and angular in shape and relatively evenly sized
- riprap should be a minimum of 0.3 m above the channel design flow level
- feather the ends of the riprap into the up and downstream banks
- voids should be filled to prevent underlying material from washing out
- specifications to include landscape preservation and water pollution prevention measures (see Vegetal Fact Sheet)

Geotextile Filter

- geotextile to be overlapped (in direction of flow) by 500 mm or 250 mm if pegged
- non-woven Class II geotextile should be used
- the upper limit for the Filtration Opening Size (F.O.S.) is D_{85} of the soil and the lower limit is 0.5 D_{85} of the soil
- if there is seepage the geotextile should be thicker than 1 mm and should be made of polyester

Drawing Requirements

- line and grade
- edge finish
- design water levels on plan and profile
- filter fabric (if applicable)
- filter fabric overlap (inset)
- areas of watercourse preservation

Specification Requirements

- stone size and type
- geotextile should not be left unprotected in direct sunlight for more than 3 days
- set and placed in stable manner
- random placement
- drop height, < 1.0 m
- Payment - riprap, m^3
  - filter - geotextile, m^2
    - granular, m^3

Design Steps

i) estimate stone size and determine permissible shear stress
ii) estimate channel slope, side slope, longitudinal slope, discharge and estimate flow depth
iii) determine Manning roughness coefficient
iv) compute flow depth
v) compare computed flow depth with estimated flow depth. If the difference is greater than 0.03 m then repeat steps (ii) through (iv). Use the computed flow
vi) calculate the shear stress for the channel bed and the side slopes. If the shear stress is greater than permissible then repeat steps (i) through (v)
vii) for channel bends:
  a) determine bend shear stress factor
  b) calculate shear stress in bend
  c) if shear stress is greater than permissible then repeat steps (i) through (vii)
  d) calculate superelevation. Check to ensure channel depth is sufficient
viii) compute water surface profile for flow rates greater than 1 m^3/s
ix) determine vertical and longitudinal extent of protection
x) determine appropriate filter material and dimensions

Disadvantages

- labour intensive installation
- vehicle access to site is required
- may not withstand high flows
- subject to vandalism
- geotextile can be undermined
- high initial cost if stone is not readily available
- if not installed properly the erosion problem will shift up or downstream
- stone to be selected in advance
- stone size may be dependent on local availability

Construction Steps

- clear and grub
- temporarily divert watercourse
- site preparation
- excavation and grading
- install seepage drains (if necessary)
- place filter layer
- place riprap and vegetative finish
- dispose of surplus material

Advantages

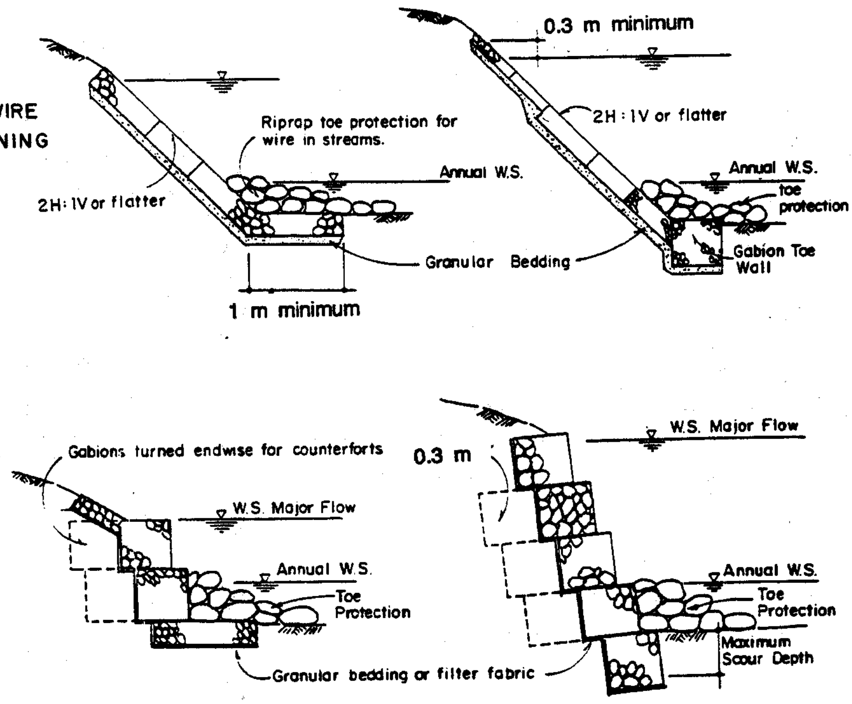
- adjusts to minor shift in subgrade
- can be placed around existing vegetation or vegetation can re-establish
- tolerates lateral seepage
- does not require heavy machinery
- easy installation
- provides habitat for aquatic organisms
- provides energy dissipation
- low maintenance, easily repaired
- provides immediate protection
- inexpensive if stone is available

Design Considerations

- vertical and longitudinal extent of protection
- edge treatment
- rock dimensions, quality
- bank slope
- filter type and dimensions
- rock layer thickness and gradation

GABION

Lined Channel



Source: Urban Storm Drainage Criteria Manual, 1968, by Wright-McLaughlin Engineers Engineering Consultants.

Section Shape

- trapezoidal
- rectangular

Typical Characteristics

- gabion baskets
  - side slopes flatter than 1.5:1
  - cross sectional area
    0.9 m x 0.3 m
    or 0.9 m x 0.5 m
    or 0.9 m x 0.9 m
- gabion matts
  - side slopes flatter than 2:1
  - cross sectional area
  - 2 m x 0.23 m
- length variable
  or 3 m x 0.23 m

Cross References

- OPSS: 201, 501, 510, 512, 514, 517, 902, 1430, 1860, various dates
- Water Resources Protection Technology, J.T. Tourbier & Westmacott, 1981
- Photographic Examples, Appendix E

Maintenance

- re-tie broken wires
- check for minor erosion

Computation Methods

- maximum permissible velocity
- maximum permissible tractive force (recommended)

Design Components

- layout of scheme
- bank and foundation preparation
- unit size and configuration
- stone size and quality
- edge treatment
- backfill and filter considerations
- basket fabrication

Lined Channel (continued)

GABION

Criteria/Guidelines

- upstream and downstream toe wall required
- specifications to include landscape preservation and water pollution measures (see Vegetal Fact Sheet)
- cutoff on upstream and downstream face
  - in steps and continuous
- seepage drain may be required
- lining blended at boundary
- line of baskets at stream level to prevent toe failure, deep enough to prevent against scour
- vehicular access required
- gabion matts
  - upper vertical limit above high water line
  - freeboard allowance dependant on velocity near mat
  - upstream/downstream ends should be protected by a cut-off wall of gabions
  - slopes flatter than 2:1 (horizontal:vertical)
- for steep slopes stack gabion baskets to form a retaining wall
- protect upstream and downstream of the works - feather into existing channel with riprap
- 30-50 year lifespan
- 70% of stone should exceed wire mesh opening size
- granular or synthetic filter material required
- lining should be stapled or secured with gabion counter forts every 3-4 m
- stakes driven at least 1.3 m
- counterforts at least 0.3 m deeper than lining

Drawing Requirements

- plan and profile
- lines and grades
- number and dimension of units
- filter fabric/lining
- water levels
- design water levels on plan and profile
- freeboard
- areas of watercourse preservation

Specification Requirements

- geotextile not left unprotected in direct sunlight for more than 3 days
- dimension and tolerances of units
- mesh opening, wire for netting and binding, inspection and testing
- packaging and marking
- payment - gabions, m^3
  - gabion structures, m^3
  - excavation, m^3
  - filter layer - geotextile, m^2
    - granular, m^3

Computation Steps

i) estimate stone size and determine permissible shear stress

ii) estimate channel slope, side slope, longitudinal slope, discharge and estimate flow depth

iii) determine Manning roughness coefficient

iv) compute flow depth

v) compare computed flow depth with estimated flow depth. If the difference is greater than 0.03 m then repeat steps (ii) through (iv). Use the computed flow

vi) calculate the shear stress for the channel bed and the side slopes. If the shear stress is greater than permissible then repeat steps (i) through (v)

vii) for channel bends:
  a) determine bend shear stress factor
  b) calculate shear stress in bend
  c) if shear stress is greater than permissible then repeat steps (i) through (vii)
  d) calculate superelevation. Check to ensure channel depth is sufficient

viii) compute water surface profile for flow rates greater than 1 m^3/s

Construction Steps

- prepare site
- disposal of surplus material
- excavate and backfill
- compacting
- place filter layer (granular or geotextile)
- assemble gabion units
- place gabion stone
- attach connecting wires
- secure lids
- granular backfill
- if required:
  - (i) topsoil and seed; or
  - (ii) concrete cap

Note: off site basket fabrication is not recommended

Advantages

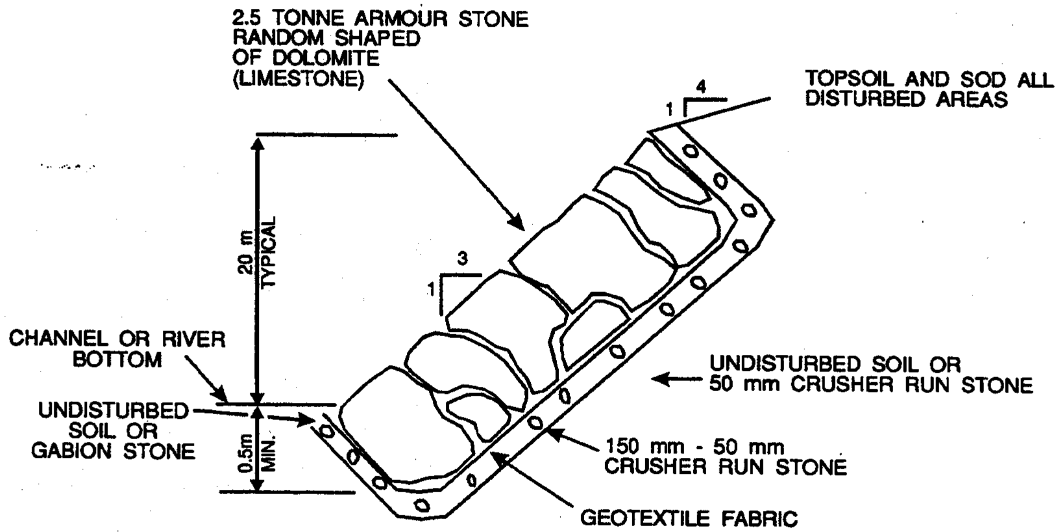
- provides dual purpose channel protection and retaining wall support
- added vegetation increases protection
- adjusts to minor shifts in the subgrade
- allows free drainage
- provides energy dissipation
- can be used for a wide range of channel dimensions

Disadvantages

- labour intensive installation
- subject to vandalism
- difficult to install in deep water
- expensive
- dewatering may be necessary
- difficult to construct in areas with limited access
- protection limited to the lifespan of the wire basket
- sometimes failure due to erosion of bed material

Lined Channel

ARMOUR STONE



Section Shape

- trapezoidal
- triangular
- parabolic

Maintenance

- fill voids created by removal of interspatial material

Typical Characteristics

- side slopes flatter than 1.5:1 (horizontal:vertical)
- high flow velocities (i.e. 3 m/sec)
- 2.5 tonne stone (dolomite or limestone)
- granular or synthetic filter

Computation Methods

- maximum permissible velocity (Manning equation)
- maximum permissible tractive force
- backwater computations for significant structures

Cross References

- OPSS: 201, 501, 510, 514, 517, 902, 1860, variouse dates
- Photographic Examples, Appendix E

Lined Channel (continued)

ARMOUR STONE

General Criteria/Guidelines

- vehicular access required
- filter required
- layer of crusher run stone required

Drawing Requirements

- plan and profile
- slopes
- filter fabric
- edge finishes
- channel bottom
- subgrade
- design water levels on plan and profile
- areas of watercourse preservation

Specification Requirements

- stone size and shape
- filter size
- geotextile not to be left unprotected in direct sunlight for more than 3 days
- landscape preservation and water pollution prevention measures (see Vegetal Fact Sheet Criteria/Guidelines)

Computation Steps

i) estimate stone size and determine permissible shear stress

ii) estimate channel slope, side slope, longitudinal slope, discharge and estimate flow depth

iii) determine Manning roughness coefficient

iv) compute flow depth

v) compare computed flow depth with estimated flow depth. If the difference is greater than 0.03 m then repeat steps (ii) through (iv). Use the computed flow

vi) calculate the shear stress for the channel bed and the side slopes. If the shear stress is greater than permissible then repeat steps (i) through (v)

vii) for channel bends:
a) determine bend shear stress factor
b) calculate shear stress in bend
c) if shear stress is greater than permissible then repeat steps (i) through (vii)
d) calculate superelevation. Check to ensure channel depth is sufficient

viii) compute water surface profile for flow rates greater than 1 m^3/s

Construction Steps

- excavation and backfill
- disposal of surplus material
- shape and compact
- place crusher run stone
- place filter layer
- place armour stone
- finish edges

Advantages

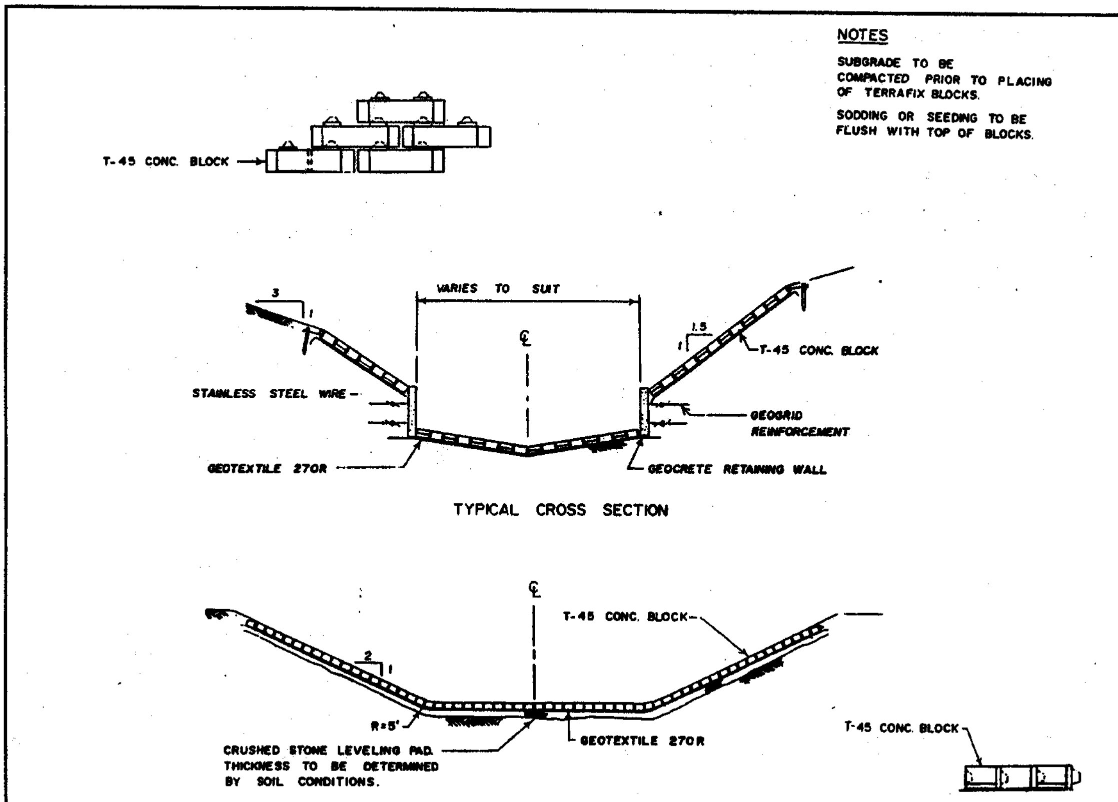
- more natural appearance than gabions
- allows free drainage
- low maintenance

Disadvantages

- requires heavy equipment accessibility
- nearby source may not be available

Lined Channel

CONCRETE BLOCK



Source: Terrafix Interlocking Concrete Block System, Standard Specification Sheets, 1989.

Section Shape

- trapezoidal
- triangular
- parabolic

Computation Methods

- maximum permissible velocity (Manning equation)
- maximum permissible tractive force
- compute water surface profile for peak flow rates greater than 1 m^3/s

Typical Characteristics

- Terrafix lining 3 components:
  - -interlocking bricks
  - -geotextile underlay
  - -reinforcing cables
- Terrafix size, weight, application:
  - -T45: 610x152x114mm; 205 kg/m^2; light to moderate flow velocities
  - -T50: 610x152x114mm; 244 kg/m^2 moderate velocities
  - -T60: 642x178x150mm; 317 kg/m^2; moderate impingement and turbulence
- Cable Concrete Mats size, weight, application:
  - -Mats: 1.22x4.88m, 36 blocks in a 3x12 pattern
    - 20cc mat: channel and pond lining for low flow velocites
    - 35cc mat: channel lining, culvert outfalls, moderate flow velocites
    - 45cc mat: river bank stabilization, shoreline revetment, moderate to high flow velocities
    - 70cc mat: areas subject to severe erosion, river bank stabilization, shoreline revetment and high flow velocities
  - -Blocks: 394x394mm (base), 292x292mm (top)

Maintenance

- refill and replant cells if soil is removed by water

Cross References

- OPSS: 201, 501, 510, 514, 517, 902, 1860, various dates
- Terrafix Interlocking Concrete Block System, Std Spec. Sheets
- Test Results for Terrafix Erosion Control Projects Inc., K. Hill, 1990
- Stability Tests of Terrafix Blocks in Channel Flow, Y.L. Lau, 1990
- Photographic Examples, Appendix E
- Guide to Design and Placement of Cable Concrete Mats, West Lorne Precast Limited, J.A. McCorquodale

Lined Channel (continued)

CONCRETE BLOCK

General Criteria/Guidelines

- subsoil must be well graded prior to placement and provide a stable base (bedding layer may required)
- seepage drain and filter fabric may be required
- anchors and reinforcing wires may be required
- top of anchor to be 100 mm below final grade
- anchor interval = 900 mm
- overlap adjacent sections of geotextile by a minimum of 300 mm
- dig outer edges of geotextile and blocks a minimum of 150 mm into the subgrade
- do not permit construction equipment on geotextile or block system
- upstream edges of block system to be rounded into subgade 600 mm and backfilled
- sodding or seeding to be flush with blocks
- pre-assembled block sections or individual blocks available

Drawing Requirements

- plan and profile
- side slopes
- block type and alignment
- filter fabric
- edge finish
- design water levels on plan and profile
- reinforcing wire
- areas of watercourse preservation
- subgrade material and depth

Specification Requirements

- compressive strength of concrete and grout
- maximum absorption
- freeze-thaw resistance
- reinforcing wire: type and material
- anchors: type and material
- geotextile: tensile strength, elongation, burst stress, permeability and F.O.S
- geotextile not to be left unprotected in direct sunlight for more than 3 days
- Payment: by m^2

Design Components

- layout of general scheme
- bank preparation, m
- mattress and block size
- slope
- edge treatment
- filter requirements
- surface treatment
- toe design
- vertical and longitudinal extent of protection

Construction Steps

- prepare site: disposal of surplus material; excavate and backfill
- bedding layer (if required)
- subgrade preparation: shape and compact
- anchors (if required): place and backfill
- place and secure geotextile
- place, interlock and secure blocks on geotextile in running bond pattern
- reinforcing wire (if required): feed through pre-formed holes; twist, trim and attach ends to anchors
- infill joints and cavities; vegetate open cells and bare slopes

Advantages

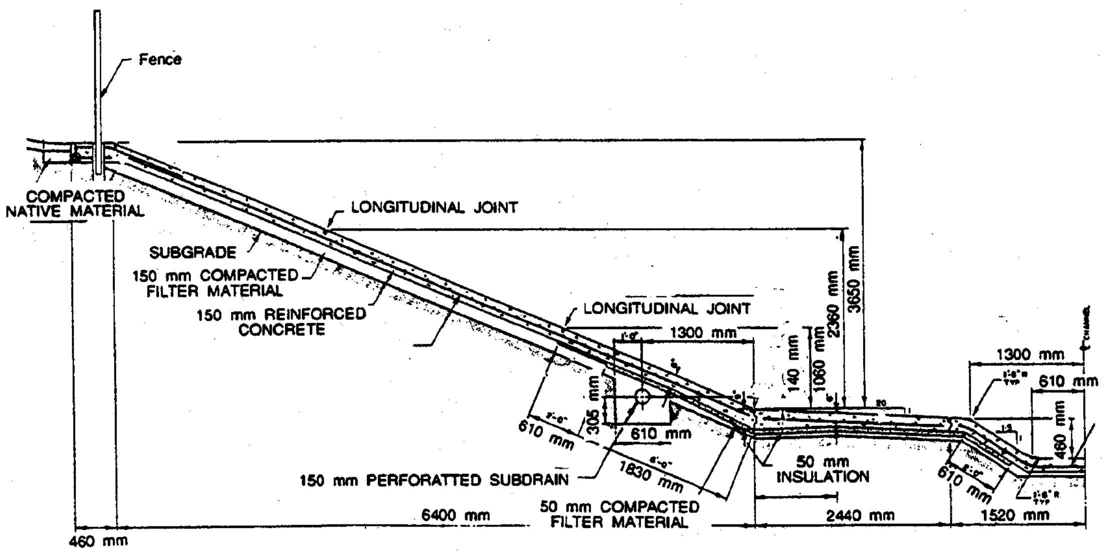
- available in various sizes
- flexible, conforms to ground contours
- somewhat permeable
- allows natural vegetation to reestablish
- minimum access requirements
- low maintenance
- available in prefabricated mat or individual blocks

Disadvantages

- extended flooding increases maintenance costs
- specialized equipment required for mat installation
- initial costs are high
- limited flexibility
- subject to failure along banks at dynamic channel bends

CONCRETE

Lined Channel



Source: Channel Improvements to Grand Marais Drain, 1974, by M.M. Dillon Limited.

Section Shape

- any channel shape that can convey flow and provide adequate freeboard
  - rectangular
  - trapezoidal
  - triangular

Typical Characteristics

- flow velocity > 4 m/s < 8 m/s
- side slopes flatter than 1.5:1 (horizontal:vertical)

Computation Methods

- maximum permissible velocity (Manning equation)
- backwater computations for significant channels

Maintenance

- check for freeze-thaw damage

Design Components

- layout of general scheme
- bank and foundation preparation
- bank slope
- pavement thickness
- pavement reinforcement
- edge treatment
- stub walls
- filter/subsurface drainage design
- pressure relief
- concrete quality

Cross References

- OPSS: 201, 501, 510, 514, 517, 902, 1860, various dates
- Photographic Examples, Appendix E

Lined Channel (continued)

CONCRETE

General Criteria/Guidelines

- minimum thickness 150 mm, thickness depends on flow velocity and soil conditions
- expansion joints and reinforcing steel should be provided
- soil tests to determine if sufficient quantities of sulphates exist to cause concrete deterioration
- provide underdrains and weep holes
- failure due to removal of foundation support by subsistence undermining, outward displacement, hydrostatic pressure, sliding or erosion
- flatter bank slopes than 1.5:1 (H:V)
- best used where bank erosion caused by high flow velocities and cost of failure is significant
- toe must be protected
- provide protection from hydrostatic uplift (i.e. perforated underdrain pipe)
- generally, contraction joints 8-15 m apart
- slabs constructed in alternate panels to account for initial placement shrinkage
- significant attention to the design of the subdrainage system

Drawing Requirements

- plan and profile
- slopes
- concrete thickness
- design water levels on plan and profile
- edge finishes
- expansion joints and reinforcing steel (spacing & size)
- underdrains, weep holes, cleanouts, cutoffs and fillets
- areas of watercourse preservation

Specification Requirements

- freeze-thaw resistance
- geotextile not left unprotected in direct sunlight for more than 3 days
- mix or strength of concrete
- landscape preservation and water pollution prevention measures (see Vegetal Fact Sheet Criteria/Guidelines)

Computation Steps

- determine channel section and Manning roughness coefficient
- estimate depth of flow
- determine hydraulic radius and cross sectional area
- calculate flow velocity
- determine channel capacity
- if channel capacity is adequate, add freeboard
- supercritical flows profile upstream to downstream computation
- uniform steady flow Manning equation
- gradually varied flow backwater profile
- compute water surface profile for peak flow rates greater than 1 m^3/s

Construction Steps

- prepare site
- disposal of surplus material
- excavate and backfill
- dewater site
- shape and compact
- prepare formwork
- place reinforcing
- pour concrete lining
- form removal

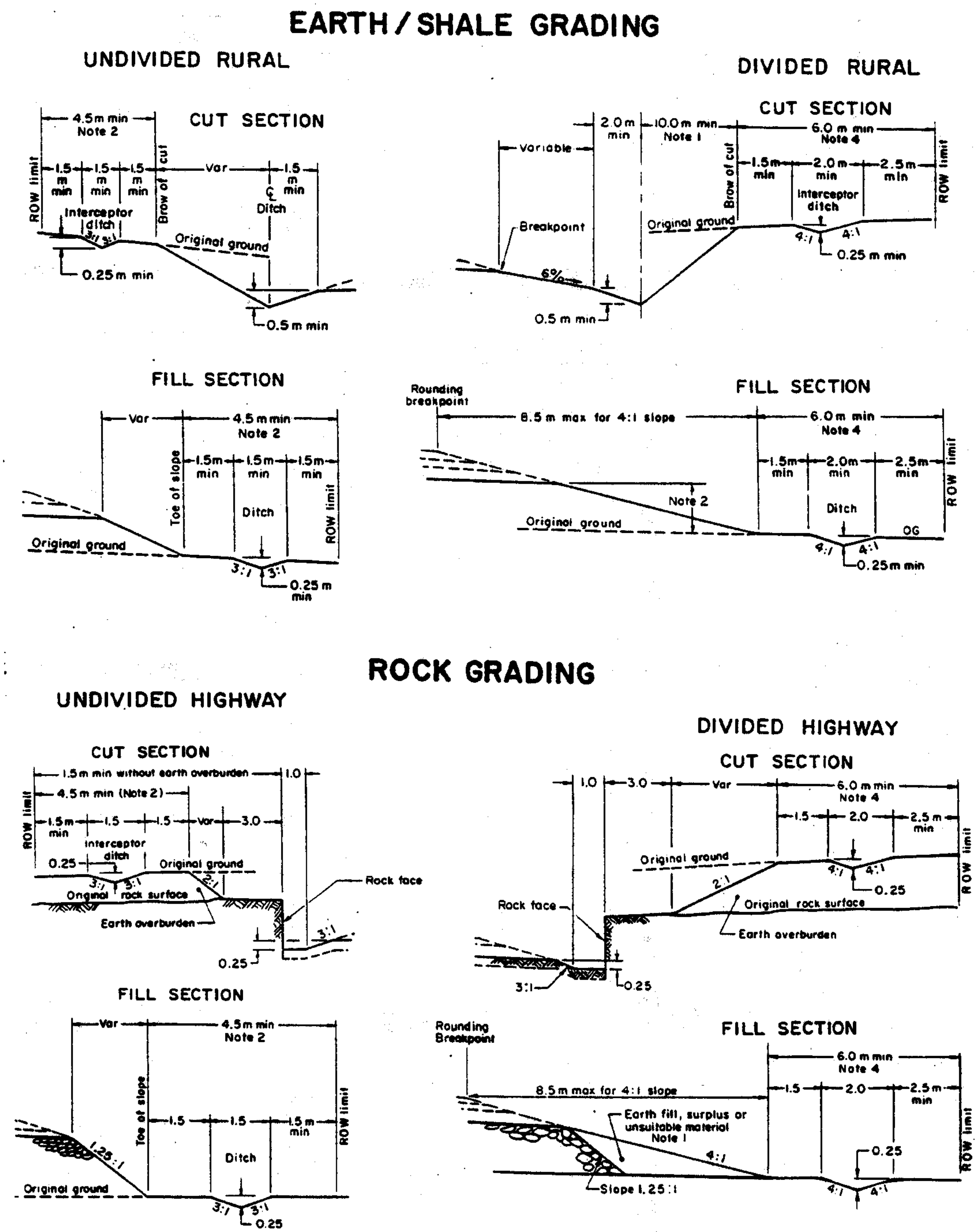
Advantages

- adaptable to existing site conditions
- relatively easy to apply
- durable
- allows steeper slopes
- low maintenance
- high hydraulic capacity

Disadvantages

- subject to settlement
- subject to freeze-thaw damage
- not aesthetically pleasing
- may disrupt subsurface drainage
- high initial costs and replacement cost
- may increase downstream flow velocities and erosion

Roadside Ditch



Source: OPSD 200.01, 200.02, 201.01, and 201.02, by Ontario Provincial Standards and Specifications, April 2, 1990.

(continued)

Roadside Ditch

General Criteria/Guidelines

- maximum flow depth 1.0 m
- minimum freeboard 0.3 m
- minimum width between top of ditch and toe of slope 1.5 m
- subdrain may be required
- ditch invert at least 0.3 m below road granular material

Drawing Requirements

- profile
- cut, fill and tangent sections
- slopes
- dimensions
- original surface lines
- subgrade and materials
- subdrain detail (if required)
- profile grade
- rounding breakpoint

Design Steps

Construction Steps

- prepare site
- excavate and backfill
- shape and compact
- line channel if required

Section Shape

- triangular
- usually 4:1 (H:V) side slopes,
  2:1 (H:V) maximum
  4:1 (H:V) minimum

Maintenance

- regrading if required
- removal of vegetation
- spring ditching

Dimensions

- varies
- minimum longitudinal slope 0.5%
  maximum longitudinal slope 5.0%

Computation Methods

- Manning equation
- compute water surface profile for flow rates greater than 1.0 m^3/s

Cross Reference

OPSD: 200.01, 200.02, 201.01, 201.02, 214.01, 214.02, 214.03, 215.01, 215.02, 215.03, 216.0, various dates

Ministry of Transportation
Ontario

MTO

Chapter 6
Temporary Sediment and Erosion Control

Drainage and Hydrology Section
Transportation Engineering Branch
Quality and Standards Division

6 Table of Contents

List of Figures

Purpose of This Chapter

The main purpose of this chapter is to provide the design considerations required for the design of temporary sediment and erosion control measures suitable for highway construction sites. It is acknowledged that design of sediment and erosion control measures during construction is a multidisciplinary endeavour and, in most cases, is undertaken as part of the construction contract. Therefore, the objective is to provide a general guide to designers on basic hydraulic considerations for the design of temporary sediment and erosion control measures.

The process required for the development of sediment and erosion control plans is not addressed in this manual. All procedural matters related to the development of a plan are covered in the *Environmental Manual*, *Erosion and Sedimentation Control*, working draft (MTO, 1994). This chapter provides technical details in support of that manual and applicable OPS standards.

Temporary soft sediment and erosion control measures, or measures not requiring the application of hydraulic design principles are covered in this chapter to the extent that they provide sufficient background information in support of other MTO manuals. They were also included to provide information where no information is available in other MTO manuals.

Permanent erosion and sediment control measures applicable to temporary works are discussed in Chapter 5.

This chapter is divided into two main sections. They are:

- General Design Considerations; and
- Temporary Sediment and Erosion Control Measures.

The first section focuses on the factors affecting erosion and sedimentation processes and the considerations required for the selection of appropriate control measures. This section provides linkage with the *Environmental Manual* (MTO, 1994). The second section covers the main scope of this chapter and discusses the technical design aspects requirements for the different temporary erosion and sediment control measures.

General Design Considerations

Erosion and Sedimentation

Flowing water is a powerful agent in removing soil from large areas of construction projects, gouging into cut and fill slopes to form rills and gullies, and causing erosion of the beds and banks of channels. The erosion and sedimentation process involves three basic steps: detachment of the soil particles, transportation of the material by flowing water, or other agents, and subsequent deposition.

In the erosion process detachment occurs mainly as a result of impact by rain drops on an exposed soil surface. Transportation is initiated when surface runoff removes particles in a relatively uniform layer by sheet erosion. As the runoff flows downhill, it increases in velocity and energy, enhancing its ability to detach and move soil particles, and small closely-spaced channels, only a few centimetres in width and depth are formed. This is called *rill erosion*. Sheet and rill erosion together are the principal area of interest in this chapter. If conditions are such that the runoff becomes concentrated in one rill, a deeper and wider channel forms by what is termed *gully erosion*. If remedial action is not taken, the whole slope may eventually become dissected by gullies.

Where the flow gradient flattens, or the velocity is reduced by other features such as vegetation buffer strips, filter barriers or sediment traps, the larger particles, such as gravel and coarse sand become deposited. The fine sand, silt and extremely fine colloidal material are carried further until they are either deposited, or enter a ditch or swale. At this point the erosive power of the flowing water is increased, and the channel is vulnerable to erosion until permanent cover can be established. Channels on moderate to steep gradients are quite susceptible to degradation and gullying, which may produce large quantities of sediment.

Deposition of all but the finest particles takes place when the velocity of the flow is checked. At the point of entry into a body of still water much of the sediment may be deposited. Extremely fine particles of colloidal clay may remain suspended in water and are often the cause of unacceptable turbidity.

General Approach to Temporary Erosion and Sediment Control

Successful control of erosion and sedimentation on construction sites depends on the adherence to simple common sense principles. These principles include:

- The area of soil exposed at any one time during construction, and the length of time it is exposed, should be minimized.

- Soil cover should be applied as soon as possible after exposure of the soil.
- Surface water runoff should be intercepted during construction and safely channelled to suitable outlets.
- Flow velocity should be maintained to below the scour velocity whenever possible.
- Sediment should be intercepted as close to its source as practically possible.
- Control measures should be carefully selected on the basis of cost, benefit, effectiveness, and practicality.
- Control measures should be installed correctly at the appropriate time, and should be properly maintained. Sediment basins, when specified, should be constructed before the start of rough grading.
- Ditches and other channels on erosive grades should be properly designed with suitable temporary erosion controls.
- The performance of temporary and permanent control measures should be monitored during construction and over the long-term, and new or improved methods developed as required.
- Close cooperation is necessary between the design, environmental, and construction groups, as well as involved agencies or parties.

Construction in Water

In addition to the considerations outlined above, the following points should be considered when construction has to be done in water (in a watercourse, lake or wetland):

- Construction in streams which support significant fish migration should be limited to periods which avoid sensitive times of the fish life cycle.
- Vegetation in designated areas adjacent to stream banks or lake shores should be preserved
- Construction equipment crossing on sensitive streams should avoid instream crossing and be limited to designated locations.
- Water from dewatering operations should be filtered or passed through sediment traps where necessary to prevent unacceptable turbidity.

Design Storm Return Periods for Temporary Structures

The sizing of sediment and erosion control structures such as ponds, ditches and chutes (permanent or temporary) depends greatly on the flow rate and hence, the design storm return period. When designing permanent facilities, general standards usually apply. However, when designing temporary structures, there are no set standards. The designer needs to assess the appropriate storm return period based on an assessment of the acceptable level of risk of damage due to a failure and the period the structure will be in use, balanced with the increased cost of constructing larger facilities to handle a design storm with a more conservative return period.

Consideration should be given, when selecting the design storm return period, to the fact that on construction sites, secondary sediment and erosion control measures may not be in place to moderate the impact of failure of a facility. Secondary measures may include, for example, a grassed ditch downstream of a sedimentation basin or a vegetated buffer strip downstream an interceptor toe drain.

Erosion and Sedimentation Analysis

Erosion analysis are conducted to assess the potential impact of a project on a sensitive environment, to investigate means by which this impact may be avoided or mitigated, to identify potential concerns to be considered in the detail design stage, and to determine the appropriate storm return period suitable for design.

The analysis may include the following:

- Identification of sensitive areas likely to be affected by the project.
- Assessment of the degree of sensitivity of the areas.
- Identification of individual construction areas likely to have an impact on the sensitive areas.
- Assessment of the potential sediment yield of the impacting areas.
- Assessment of the potential impact of the proposed construction, based on an integration of the sensitivity and sediment yield ratings.
- Determination of the acceptable level of risk and appropriate design storm return period.
- Selection of mitigation measures which can be implemented.
- Identification of concerns which should be studied further.

Sediment and erosion control on construction sites is a multidisciplinary activity that requires the consideration of environmental, hydrologic, geotechnical, construction and economic issues, among others. This section focuses on these issues from the hydrologic and hydraulic perspective. It covers technical aspects related to selection and design of temporary sediment and erosion control measures requiring hydrologic and hydraulic calculations. Therefore, a detailed procedure for the selection of control measures and sensitivity/impact evaluation is not provided. Instead, a brief discussion is presented to highlight the environmental aspects and the circumstances in which a project may impact the environment.

Identification of Sensitive Areas

Environmentally Sensitive Areas

Environmentally sensitive areas are normally identified from knowledge of local environmental conditions. They may be identified by MNR, MOEE, conservation authorities, private interest groups such as fishing or naturalist clubs, and individuals. In all cases, the designation of a sensitive area must be justified by supporting information and documentation. The validity of this designation can be checked using information obtained through field investigations, remote sensing studies, maps, literature reviews and other sources.

The degree of environmental sensitivity of an area (high, medium or low) is one of the factors used in assessing the potential impact of the project. If the sensitivity rating is based on the opinion of the agency or group originally identifying the area, its validity may need to be reviewed. The degree of sensitivity may be assessed from consideration of the factors discussed below. Consultation with a fisheries biologist for such assessments is recommended.

Sensitivity of Receiving Waters The environmental sensitivity of lakes and streams may be judged from the fish species present or likely to be present, taking into account the probability of removal of existing stream obstructions such as dams. Cold-water streams (in which brook, rainbow or brown trout live, spawn or migrate) are rated as highly sensitive. Those with warm-water game (fish such as walleye and small-mouth bass) are rated as moderately sensitive, and those with coarse species (fish such as carp) have a low sensitivity. The latter streams have warm water and generally a high turbidity.

Wetlands, including marshes, swamps, bogs and fens, are frequently sensitive to sedimentation, which may seriously affect their value as habitat for fish, wildlife and plant species and as pollutant and nutrient filters. Their sensitivity is therefore, rated according to their value with respect to these functions.

Sensitivity of Downstream Uses Downstream uses include recreation, water supply and water engineering facilities, which include agricultural drains, reservoirs, canals and harbours. The degree of sensitivity depends on the type of facility and its distance from the construction site. For example, a domestic water supply provided with minimal treatment and located close to the site would be rated as highly sensitive. At the other extreme, an advanced water treatment plant may have a low sensitivity.

Sensitivity of Terrestrial Areas Terrestrial environments sensitive to sedimentation include natural areas such as woodlands and sensitive or rare plant communities, and other areas such as market gardens, sensitive farmland, agricultural research areas, private lawns and gardens. These are generally identified during field inspections and from discussions with other ministries and the public.

Sediment Yield

The impact of a project depends on both the degree of sensitivity and the potential sediment yield of the site.

The first step in assessing the yield is to identify areas of construction which will contribute sediment to the sensitive area. The sediment yield can be assessed qualitatively, as discussed in the section below, or quantitatively using mathematical or lumped models such as the Universal Soil Loss Equation (USLE) or the Modified Universal Soil Loss Equation (MUSLE). A detailed theoretical review of the USLE and MUSLE is covered in Chapter 8. In most cases, a quantitative analysis may not be warranted for the design of temporary erosion and sediment structures. However, it may be necessary to provide a quantitative assessment of the sediment yield to calculate the storage capacity of a sedimentation basin, if one is deemed necessary. The application of the MUSLE is covered in this chapter in the design example for sedimentation basins.

Assessment of Sediment Yield

The four main factors controlling the sediment yield of a site are the soil erodibility, the length and steepness of the slopes, the area of soil exposed, and the characteristics of the sediment delivery system. Further factors considered in the publication by the Environmental *Office Guidelines for Analyzing Sensitivity of Surface Water to Erosion and Sedimentation*, (MTC, 1980) are the timing and duration of the soil exposure. These are difficult to predict, and in any case, are less significant than the other factors, therefore, they are not discussed here. The overall sediment yield may be assessed qualitatively using a rating system, in the Environmental Office publication (MTC, 1980).

Soil Erodibility The sediment yield of a soil may be judged from its erodibility factor K. (See Design Chart 6.1). As the K value increases, yield increases.

Slope Length and Steepness The sediment yield of an earth surface depends largely on its length and steepness. For example, 2:1 slopes longer than 2.5 m are considered to have a high yield while 8:1 slopes shorter than 8.5 m have a low yield. (Design Chart 6.3).

Area of Exposure The amount of sediment produced by a site is proportional to the area of soil exposed at any given moment. Small areas (less than 0.5 ha) are considered to have a low rating and large areas (exceeding 1.5 ha) are rated as high.

Sediment Delivery The quantity of eroded material delivered to a sensitive area depends on a large number of factors. If the runoff passes through a well-vegetated buffer zone, the sediment delivery potential is low, whereas if it flows through a short length of bare ditch, the potential is high.

Acceptability of Sediment Delivery

Calculation of the sediment delivery rate is followed by an assessment of its acceptability in relation to the downstream environment. If the sediment passes into a stream, this assessment may be based partly on consideration of the water quality and natural sediment load of the stream. If it passes into a lake, wetland or sensitive terrestrial environment, the acceptability of the sediment delivery and controlling measures are negotiated with regulatory agencies.

Potential Impact

The potential impact of a construction site is assessed from an integration of the environmental sensitivity and sediment yield ratings. The impact may be rated as shown in Table 6.1 or by the more complex system suggested in the environmental Office publication (MTC, 1980).

Table 6.1: Potential Downstream Environmental Impact of Construction Sites

Environmental Sensitivity	Sediment Yield	Potential Impact
H	H	H
	M	H
	L	M
M	H	M
	M	M
	L	L
L	H	M
	M	L
	L	L

Temporary Sediment and Erosion Control Measures

Temporary erosion control measures perform one of the following functions:

- reduces the impact of rain drops on exposed soil;
- binds the soil particles together;
- prevents uncontrolled flows down slopes;
- reduces the velocity of sheet and channel flows;
- protects soils from high flow velocities; and
- protects soils against the impact of ice floes, debris or waves.

Temporary sediment controls function mainly by impounding the water until most of the sediment has settled out. To some extent, filtering of the sediment may also occur. These two types of control may be used in conjunction with one another.

It should be noted that several of the temporary erosion control measures are quite effective for intercepting sediment on a small scale. Examples are mulches, slope benches and interceptor ditches. Therefore, the discussion of these common measures will be covered under the same headings. It is important to note that, effective sedimentation control begins with effective erosion control.

Temporary erosion and sediment control measures can be classified into seven classes as follows:

- Vegetative measures
- Non-vegetative cover
- Runoff controls
- Check dams
- Slope modification
- Sediment traps and basins
- Construction in the dry
- Sediment barriers and filters

Where design dimensions are mentioned in the following discussion, they are intended to serve as a guide and should be modified to suit site conditions where necessary. The requirements of sediment and erosion measures may be dictated by end-result specifications rather than requirements for specific measures.

Vegetative Measures

Vegetative measures are effective temporary means for controlling erosion and sedimentation on construction sites. Vegetative methods are applied as cover on exposed soil and channels, and as buffer strips to protect adjacent areas (water and land). This section provides a brief description of temporary vegetative measures. Permanent measures are covered in Chapter 5.

Vegetative Cover

The form of vegetative cover most commonly used is grass turf, usually grown from seed but, in some situations, laid in the form of sod. Turf should be established, as soon as possible, on areas disturbed by construction or where temporary cover is needed. Vegetative cover can be original vegetation or may be established by several methods, including:

- seeding alone (rarely used),
- seeding and straw mulching,
- seeding and hydraulic mulching,
- seeding plus erosion control blanket, and
- sodding.

Vegetative cover for special situations may be provided by legumes, trees, and shrubs. For more comprehensive description of turf establishment methods for temporary and permanent cover refer to *Turf Establishment Manual* (MTO, 1992).

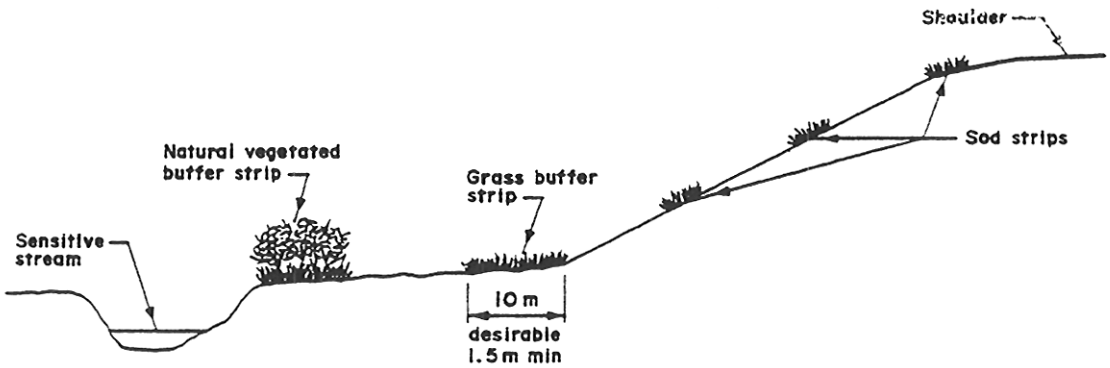
Vegetated Buffer Strips

Buffer strips of dense, well established grass or other vegetation provide an economical and easily maintained form of sediment control. Typical locations where buffer strips may be beneficial are:

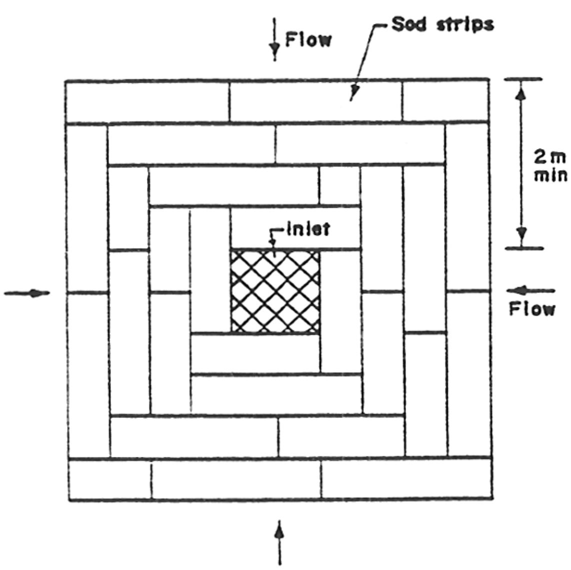
- along the top of bank of a sensitive stream (Figure 6.1(a));
- around ditch inlets (Figure 6.1 (b));
- along the edge of the right-of-way adjacent to a sensitive area;
- along the toe of an embankment;
- along the contours at intervals down an erodible slope; and
- in ditches immediately before they enter sensitive streams.

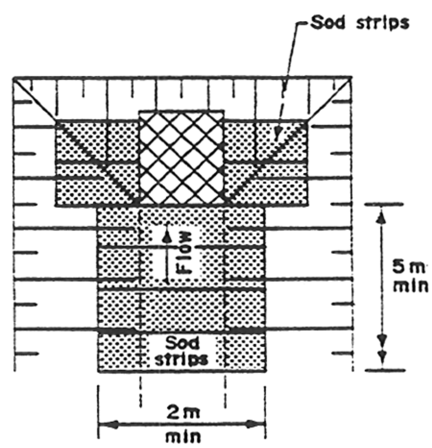
The effectiveness of a buffer strip depends on its width in the direction of flow, the density of the vegetation and the size of sediment transported. Colloidal material is not likely to be fully intercepted. The minimum width recommended is 1.5 m, but a greater width should be provided wherever possible, especially next to a stream.

Figure 6.1: Typical Uses of Vegetated Buffer Strips to Intercept Sediment



(a) Longitudinal buffer strips.





(b) Sod buffer strip for ditch inlet with flow from all directions.

(c) Sod buffer strip for ditch with flow from one direction.

(MTO, 1985)

Non-Vegetative Cover

This type of cover is generally more costly than vegetative cover, and is therefore, used only in special situations and for limited areas.

Gravel Sheeting

A blanket of coarse granular material placed on a steep or highly erodible slope, or on a slope where vegetation cannot be established, provides an effective but relatively costly form of erosion control. The blanket can also be used with filter cloth to control the flow of groundwater from a cut slope and preventing development of a seepage surface which could result in sloughing of the slope. In this application the granular layer is usually termed a blanket drain or drainage blanket, and may range from 0.15 to 0.6 m in thickness.

Gravel sheeting or blanket should be placed as soon as an excavation has been completed. The material should not be compacted.

Riprap

Riprap consists of a layer of rocks, stones or suitably sized broken concrete (100-200mm, as specified by OPSS) placed on a slope to prevent erosion. An underlying filter of geotextile or gravel is often necessary to prevent loss of the soil beneath. Riprap is used more for channel protection than for the control of sheet and rill erosion, and is therefore discussed in more detail in Chapter 5.

Riprap used on slopes not exposed to channel flow can be of a small size, in which case it may fall into the gravel sheeting category. The use of loose riprap for ground cover appears undesirable wherever there is a risk of human disturbance. Grouted riprap is sometimes used to overcome this problem, but lacks flexibility, and settlement of the underlying soil can produce unsightly cracking, settlement and cavities.

Gabions

Gabions are rock-filled wire baskets generally used for channel protection and for the construction of retaining walls rather than for ground cover. However, they are sometimes more suitable than riprap for the latter purpose since they are less subject to disturbance and are more flexible than grouted riprap.

Runoff Controls

As most sheet and rill erosion is caused by flowing water it is essential that overland flow be controlled at all stages of the project, especially before the soil cover is applied. This may be achieved by intercepting sheet flow and safely conveying it to a satisfactory outlet, or in some cases, by storing it. It is important to note that overland flow also includes all flows from external areas. These flows should be diverted from the construction site where practical.

Interceptor Dikes, Berms and Drains

Flow interception by dikes, berms and drains provides an important and inexpensive means of reducing slope erosion.

Common Practice:

- The drainage area should not exceed 2 ha (U.S. Dept. of Ag., 1975).
- Berms should have a top width and height of at least 0.5 m, and side slopes of 2:1 or flatter (Figure 6.2(a)).
- Interceptor swales and ditches should have positive drainage with a longitudinal slope of at least 1% (U.S. Dept. of Ag., 1975). This is to reduce infiltration and bank sloughing.
- Linings should be provided for channel slopes steeper than 5% and for flatter slopes in highly erodible soils.
- Long dikes should have drainage outlets at intervals to prevent a large build-up of flow.
- Drainage outlets should be protected against erosion as required.
- Safety provisions may need to be provided in case an interceptor does not function as required.

Types of Interceptor

Interceptor Berms at Tops of Fill Slopes Temporary berms may be constructed at the top edges of an embankment to funnel water from the roadbed to a chute or inlet and prevent it flowing uncontrolled down the embankment face. (Figure 6.2(b)). Sand sausages of material such as burlap are useful for this purpose in the final stages of embankment construction.

Interceptors at Tops of Cut Slope Interceptor ditches or berms at the tops of cut slopes are essential where there is a risk that overland runoff from an upland area could flow down the slope. (Figure 6.2(b)). They are often needed as permanent drainage features. This type of interceptor should be constructed before earthmoving starts.

Interceptor Channels on Slopes These are usually an integral part of slope benches. They are formed by sloping the benching down toward the inner slope face to form a definite swale at least 0.3 m deep to intercept runoff and prevent overflow onto the lower slope.

Transverse Interceptors on Road Grade A low temporary earth berm constructed across a graded roadbed can be used to intercept runoff flowing down a long grade without unduly impeding construction traffic. (Figure 6.2(c)). The flow must be diverted into a temporary or permanent chute or spillway or other outlet. Gravel or crushed rock may be used instead of earth to construct the berm, to allow some of the runoff to filter through while retaining the sediment.

Toe Drains A toe drain, ditch or berm is used to intercept runoff from a cut or fill slope and convey it to an outlet (Figure 6.2(b)). The channel should be protected, if necessary, to prevent erosion.

Toe drains for fill slopes are useful for intercepting flow before it can enter a sensitive area. However, in many cases it is desirable to allow the water to run off as sheet flow, especially where vegetation is present to induce sedimentation.

If a toe drain is necessary, it should be constructed as early as possible during rough grading in order to intercept sediment draining from the slope.

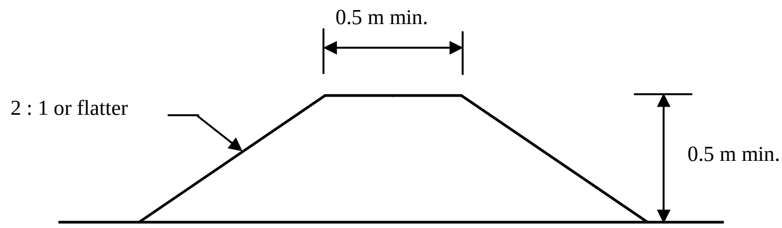
Interceptors for Groundwater Flows Groundwater is the cause of many types of slope erosion and failure, including slumping, sloughing, bank undermining, sliding of topsoil and sod, failure of mulches, and direct erosion by flowing water, either below the soil surface or after it has emerged in the form of springs.

Occasionally erosion and sedimentation problems may be avoided if groundwater can be intercepted by subdrains, wells and other methods, and conveyed through or around the disturbed area. Consult a groundwater expert where necessary.

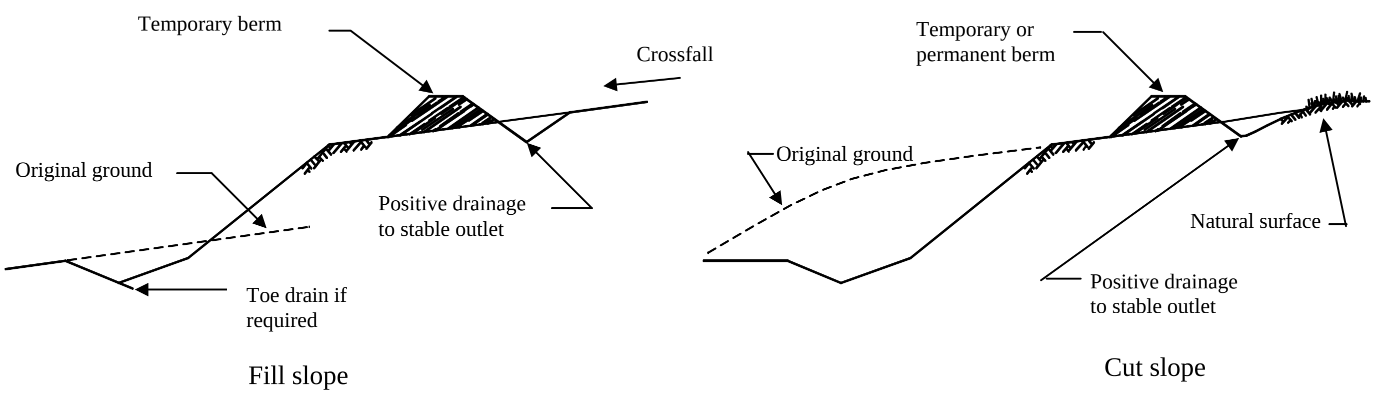
Cut-To-Fill Transitions

The control of runoff at a transition from cut to fill requires special attention during earthmoving operations because of the constantly changing grade. (Figure 6.3). This location may be subject to gully erosion because the concentrated runoff from the cut slope ditch must flow down the relatively steep gradient along the toe of the fill. One solution is to line the temporary channel with plastic sheeting. The liner should extend 1.5 m upstream from the change of slope, with the upper end buried 0.5 m to prevent the water from getting under it; it should be securely anchored with staples or other devices. The liner may have to be moved as the earthmoving progresses.

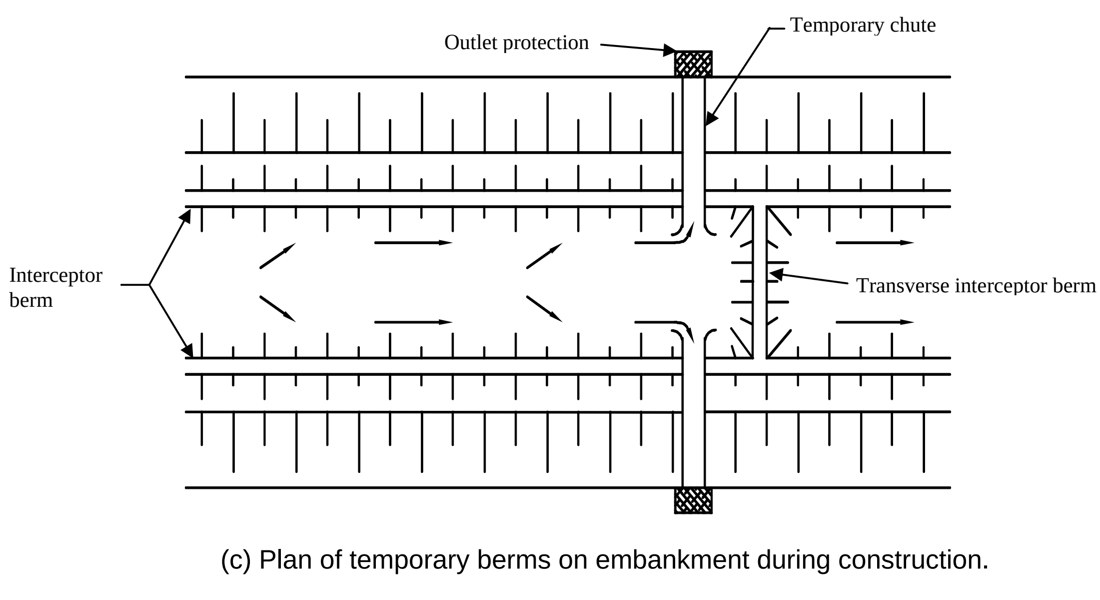
Figure 6.2: Typical Interceptor Berms and Ditches



(a) Typical section of temporary interceptor berm



(b) Interceptors for fill and cut slopes.



(c) Plan of temporary berms on embankment during construction.

Flow Spreaders

A flow spreader or level spreader is a device for converting channel flow into non-erosive sheet flow while at the same time intercepting sediment. The most effective type consists of a V-shaped trench excavated in natural ground perpendicular to the line of fall with the downstream lip exactly horizontal, as shown in Figure 6.4. Construction of a level spreader in the form of a berm is not recommended.

Flow spreaders are relatively inexpensive, but are not widely used in Ontario, possibly because of the difficulty of maintaining them adequately and the problems caused if they wash out.

Figure 6.3: Typical Temporary Drainage at Cut-to-fill Transition

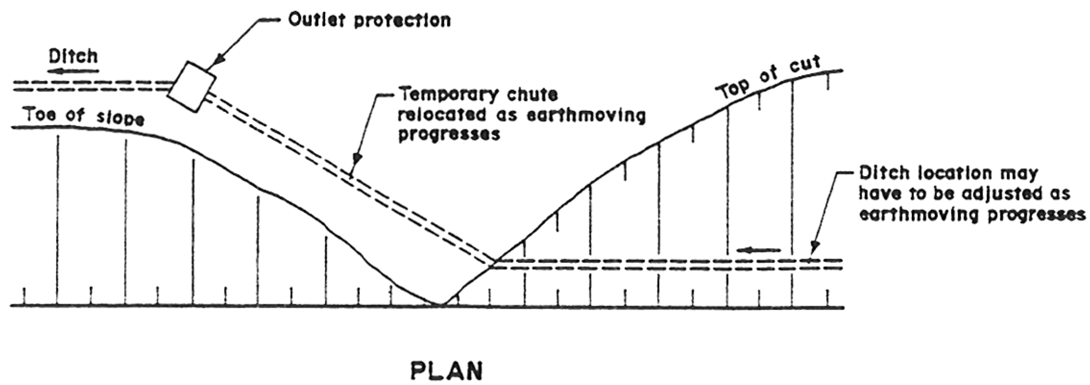
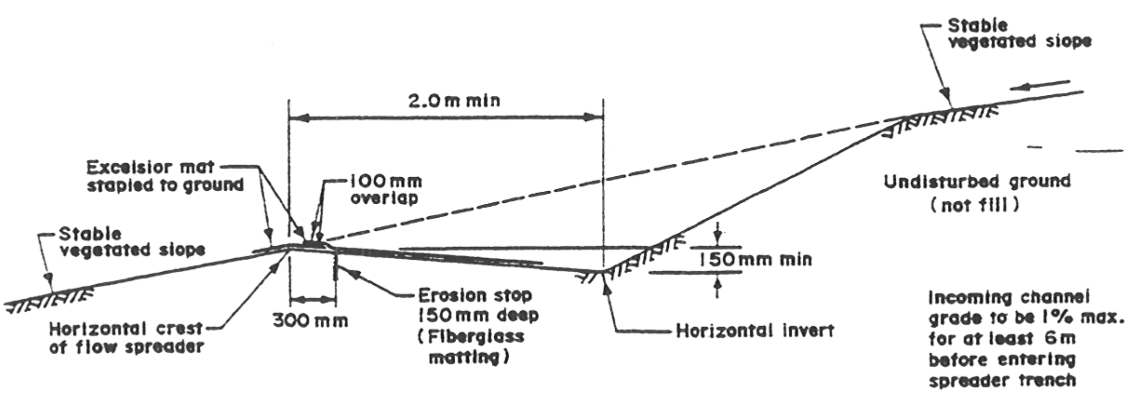


Figure 6.4: Typical Evacuated Flow Spreader



Temporary Chutes or Spillways

To avoid the formation of gullies, runoff should not be allowed to flow uncontrolled down a cut or fill slope. To ensure this, the runoff must be intercepted and diverted to a suitable outlet using interceptor dikes, berms or drains; this outlet in many cases should be a temporary or permanent chute or spillway capable of safe conveying the flow down the steep slope. Temporary chutes are described in this subsection, while permanent chutes are discussed in Chapter 5.

Chutes consist of three parts: the inlet, the chute proper, and the outlet. (Figure 6.5). The inlet must funnel the flow into the chute proper without overflowing or eroding. The chute must convey the flow without overflowing or being damaged by the high velocities. The outlet should incorporate measures for minimizing erosion. Chute inlets may be prefabricated flared end sections, riprap, or, for small flows, make shift arrangements such as a plastic liner with the upstream end buried 0.6 m.

There are two types of chute, closed pipe and open channel.

Pipe Chutes

Temporary pipe chutes may be constructed with corrugated steel, plastic or collapsible pipes as shown in Figure 6.5(a).

General Requirements:

These requirements are based on the publication *Standards and Specifications for Soil Erosion and Sediment Control in Developing Areas* (U.S. Dept. of Ag., 1975).

- Maximum drainage area of 2 ha.
- The entrance section and pipe inlet should have a slope of at least 3% toward the outlet.
- The top of the dike should be at least 0. 3 m higher than the top of the pipe.
- The inlet section should be corrugated metal pipe with watertight connecting bands.
- Flexible tubing for the chute portion should be the same diameter as the inlet pipe, securely attached to it, and anchored to the ground at 3 m intervals.
- Erosion protection should be provided at the outlet. This may consist of properly anchored sod, straw bale or sandbag barriers, or riprap and gabions for larger installations. The erosion control measures may have to be supplemented by a sediment trap if erosion cannot be controlled at a reasonable cost. (See the section on Sediment Traps).
- The soil under the entrance section and pipe inlet should be well compacted.
- The installation should be inspected and maintained, especially after storms.
- Suggested dimensions are listed in Table 6.2. However, pipe dimensions should reflect local conditions and can be calculated using the same methodology in Chapter 5 for the design of a permanent inclined drop.

Table 6.2: Dimensions of Temporary Pipe Chute

Drainage Area (ha)	Pipe Diameter (mm)
0.2	300
0.8	500
1.4	600
2.0 (maximum)	750

Source: (U.S. Dept. of Ag., 1975)

Open Channel Chutes

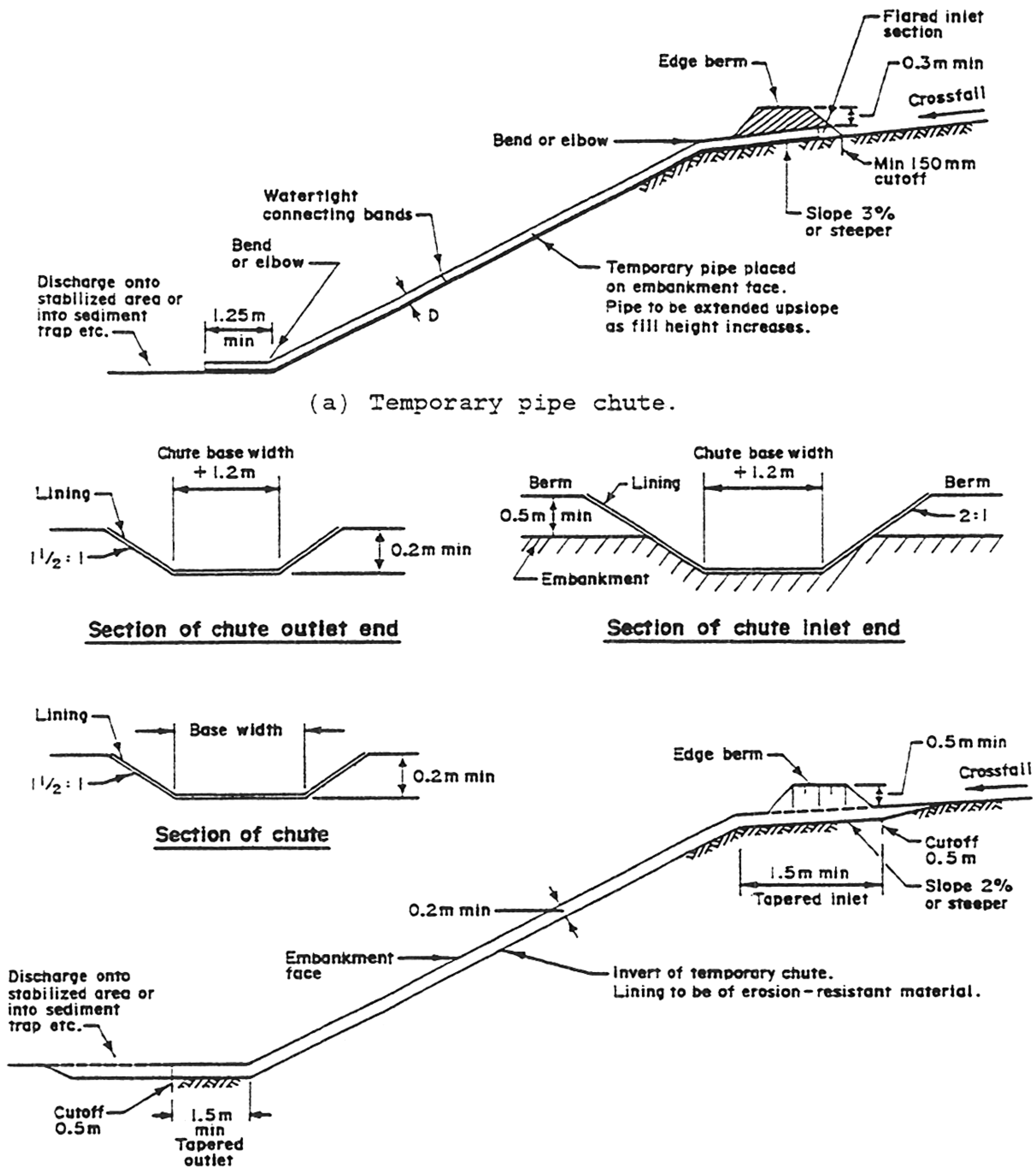
A typical open channel chute is shown in Figure 6.5(b). A wide variety of materials may be used for small temporary chutes, including plastic, plywood or metal sheeting, half-pipe sections, or sod or excelsior blanket where flow velocities will not be high. On larger chutes a bituminous lining may be required if the cost can be justified.

General Requirements:

These requirements are based on (U.S. Dept. of Ag., 1975), Size Group A for drainage areas up to 7 ha. For larger drainage areas, see Size Group B in the same publication.

- The entrance section should be at least 1.5 m long, with an inlet width 1.2 m greater than the chute base width, and should have a slope at least 2%.
- The dike or berm should be at least 0.5 m higher than the entrance channel base, with 2:1 side slopes.
- The channel lining should extend to the top of the adjacent dike.
- The chute depth should be at least 0.2 m, with side slopes no steeper than 1.5:1.
- Construction of the lining should start at its lower end.
- Cutoffs should be provided at the upper and lower ends of the lining and integral with it. The depth of cutoff should be at least 0.5 m.
- The outlet section should be at least 1.5 m long, tapering out to a width 1.2 m greater than the chute base width.
- Erosion protection should be provided at the outlet.
- The chute should be placed on undisturbed soil or well compacted fill.
- The chute slope should be no less than 20:1 and no more than 1.5:1.
- Suggested chute widths are listed in Table 6.3. However, chute dimensions should reflect local conditions and can be calculated using the same methodology in Chapter 5 for the design of permanent open channel chutes.

Figure 6.5: Typical Temporary Chutes or Spillways



(a) Temporary pipe chute.

(b) Temporary open channel chute.

Table 6.3: Dimensions of Temporary Open Channel Chutes

Drainage Area (ha)	Chute Bottom Width (m)
2	0.5
3	1.0
4.	1.5
5	2.0
6	2.5
7	3.0

Source: (U.S. Dept. of Ag., 1975)

Erosion Control at Temporary Outlets

Outlets from temporary chutes, drains and channels should be protected against erosion. This may be done by providing an erosion-resistant pad, a short channel lining, a basin to dissipate the energy, or one of many other energy dissipating devices. These may be backed up by a sediment barrier or trap to intercept any remaining sediment.

Various materials may be used for outlet protection, but the installation should not be unreasonably costly or difficult to remove after it has served its purpose. Some of the more common types are:

- sodded grass or excelsior outlet pad for fairly low velocities;
- coarse gravel, riprap or gabion pad for higher velocities;
- plastic liner; and
- sandbag or straw bale basin or barrier.

Temporary Runoff Detention for Quantity Control

Where the topography is suitable and sufficient space is available, overland runoff may be stored in a temporary detention pond. If a dam is used, care should be taken that it will not wash out and safety issues are considered to ensure no downstream damage.

Check Dams

A check dam (Figure 6.6), is a small dam constructed in a ditch or swale to reduce the hydraulic gradient and flow velocity, thereby minimizing erosion of the channel and preventing degradation and gullying. Both permanent and temporary check dams need careful design to avoid failures and related problems.

Temporary check dams are used to reduce channel flow velocities until a grass lining has become fully established. They may also intercept and store sediment, especially when used in combination with a sediment trap. Materials employed include straw or straw bales, sandbags, rock fill, gabions with filter fabric, and wood planks.

Temporary check dams suffer from the major disadvantage that they are very vulnerable to washouts caused by end-cutting, undercutting and hydrostatic pressure, and if they fail, they may cause more sedimentation downstream than would have occurred without them. This problem may be overcome to some extent by careful design, construction and maintenance, and by avoiding their use in unfavourable circumstances.

General Requirements:

- The spacing of the dams should be such that the crest of one dam is approximately level with the ditch bed immediately below the next dam upstream.
- Temporary check dams should normally not exceed 0.5 m in height at the centre of the channel.
- Check dams may be supplemented by sediment traps excavated immediately upstream.
- It is recommended that accumulated sediment be removed when its depth reaches half the height of the centre of the dam. Any damage or signs of erosion should be repaired immediately. Special care should be taken toward the end of the useful life of the dam material, which is usually 3 to 6 months.
- Temporary check dams must be removed when no longer required, and the channel reinstated to the standard of the adjacent channel.

Check dams for sediment control should be designed on the same principles, but may consist of a single row of bales if flows are likely to be very small. They require careful design and maintenance to minimize failures.

Straw Bale Check Dams

These consist of a single row of straw bales placed across a swale or ditch, possibly supplemented by a second row across the channel base. (Figure 6.6(a)). The following requirements in addition to the general requirements should be closely followed to reduce the possibility of failure.

Additional Requirements:

- This type of check dam should be installed only in channels having relatively flat gradients, small discharges and low velocities. An upper limit of discharge of 0.03 m^3/s is suggested by Sherwood and Wyand (U.S. Transp. Research Board, 1979).
- The bales must be placed in a trench at least 0.10 m deep. The upstream side must be backfilled as shown and compacted to a height of 0.1 m above the bed.

- To minimize the possibility of washouts at the ends of the dam, the bottom centres of the end bales must be at least as high as the tops of the centre bales. (See Figure 6.6(a)).
- Each bale should be securely anchored by two stakes or steel pins.
- The second (partial) row of bales should be placed on the downstream side with the individual bales staggered from those in the first row. This second row may be omitted in small swales carrying a negligible storm runoff.
- The bales must be tightly pressed together, and any gaps tightly stuffed with straw.
- If the channel bed material is highly erodible, an apron should be provided extending at least 1 m to 2 m from the downstream face. For small flows this may be of strong plastic sheet anchored under the bales. In other cases riprap or other suitable material may be required.

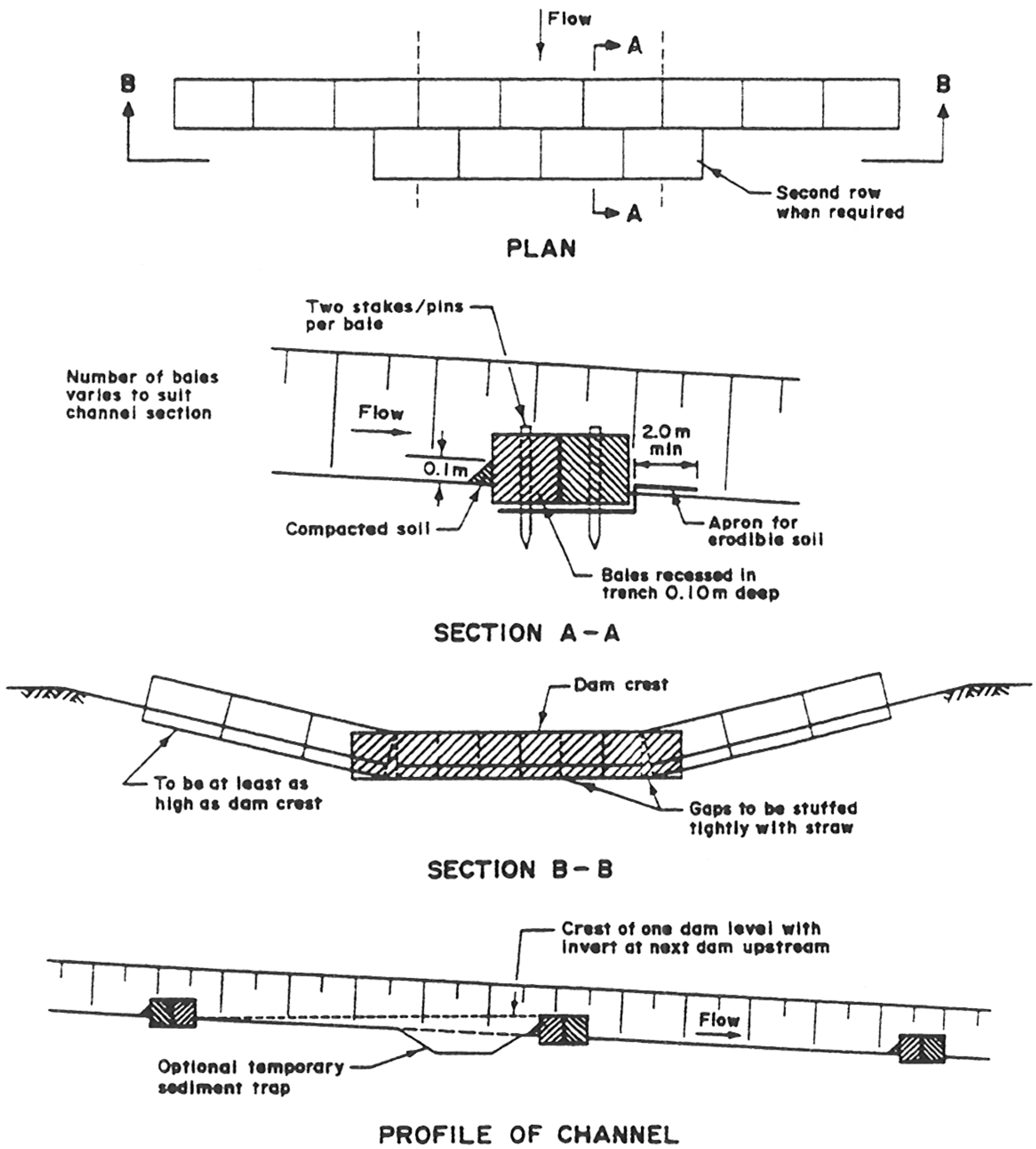
Sandbag Check Dams

Experience with sandbag check dams differs widely. Some reports indicate that the top bags tend to be pushed over, While other reports indicate that they are somewhat more effective than straw bale dams because they better resist displacement by water pressure and flotation. Regardless of these findings, sandbag check dams need careful design, installation and maintenance, and should not be used as permanent dams. The following requirements are in addition to the general requirements.

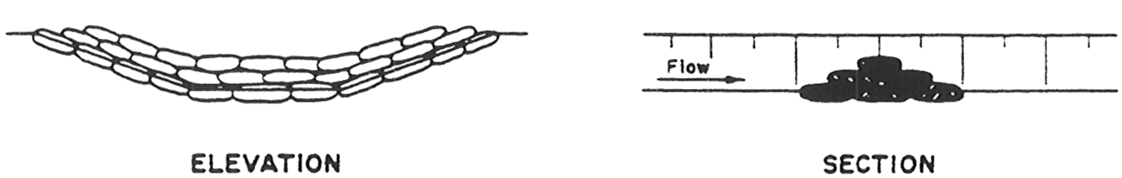
Additional Requirements:

- The sandbags should be placed in three layers, the lowest 3 bags wide, the next 2 bags and the top 1 bag wide, with all joints staggered.
- The bags should be laid in a trench 0.10 m deep to minimize undermining.
- To reduce the possibility of washouts around the ends, the bottoms of the end bags in the bottom layer should be at least as high as the top of the dam at the centre.
- The sandbags must be pressed tightly together.
- If the bed is highly erodible, an apron should be provided as described in the last bullet of the requirements for straw bale dams.

Figure 6.6: Temporary Check Dams



(a) Temporary straw bale check dam.



(b) Temporary sandbag check dam.

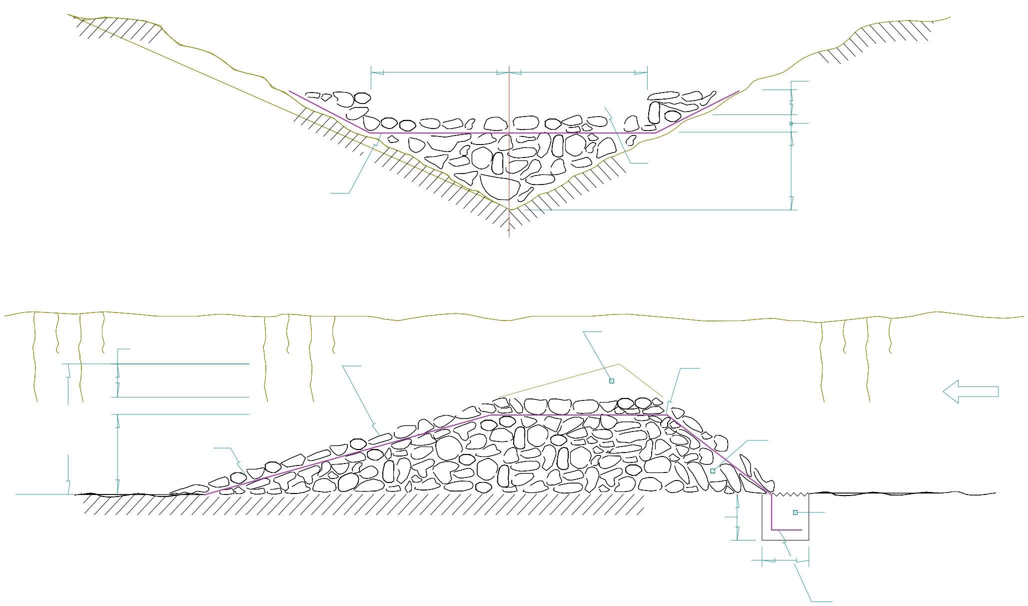
Rock Fill Check Dams

These are constructed using any rock, riprap or broken concrete available on the site (Figure 6.7). The type of material selected should be able to withstand the flow of water. The following requirements are in addition to the general requirements.

Additional Requirements:

- The rock should be piled up with a maximum upstream slope of 1.5:1 and a maximum downstream slope of 4:1.
- The rock should be placed in two layers separated by geotextile with the first layer piled across the ditch or channel to a hight of 0.45 m.
- The second layer of rock should be piled to a minimum depth of 0.1 m, anchoring the geotextile and forming a spillway centred around the lowest portion of the ditch or channel.
- The side of the ditch or channel should be protected by extending the rock pile to a height of 0.7 m on each side of the spillway.
- The depth of the spillway should measure 0.15 m from the top of the spillway.
- The width of the spillway should be 0.3 m on either side of the centre of the ditch , for V-shaped ditches, and the greater of 0.3 m or the point where the side slopes meet the bottom for a trapezoidal channel or ditch.

Figure 6.7: Typical Temporary Rock Check Dam



Ontario Provincial Standard, OPSD- 219.210

Slope Modification

Slope erosion can be reduced to some extent by flattening the gradient or by reducing the effective slope length by breaking it up into segments with benches or serrations. This reduces the amount of runoff travelling down-slope, reduces the flow velocity, increases infiltration and intercepts some of the sediment before it reaches the toe-of-slope. Most forms of slope modification require a greater width of right-of-way, and may be uneconomical where land is expensive.

Slope Flattening

Flattening a slope also increases its length and face area, and is therefore not as advantageous as it first appears. However, it may be attractive when there is an adequate width of right-of-way and sufficient material for flattening the fill slopes, or where flattening the cut slopes would provide needed borrow material.

Slope Benching

Benches are large ledges, usually 2 m to 6 m wide and at vertical intervals of say 5 m, excavated along the contours of slopes to reduce the effective slope length, slow the runoff velocity, increase infiltration and intercept sediment. (Figure 6.8). Benches should be sloped down toward the inner slope face to form a definite swale at least 0.3 m deep to intercept runoff and prevent overflow onto the lower slope.

Aside from the additional excavation and right-of-way width required, a potential disadvantage of benching in some situations is that infiltration into the bench may cause bank sloughing. Positive drainage to a safe outlet must therefore be ensured by providing a longitudinal slope of at least 1%, and allowance must be made for sedimentation when deciding the dimensions of the swale.

Slope Serration

Closely spaced steps or serrations in soft shale or other easily ripped bedrock act in the same way as benching (Figure 6.9). The steps facilitate the establishment of vegetation, and in some cases may be gradually transformed to a uniform slope by weathering and sedimentation. They are constructed parallel to the contours. Typical dimensions of the steps are 1.2 m by 0.8 m vertical.

Figure 6.8: Details of Slope Bench

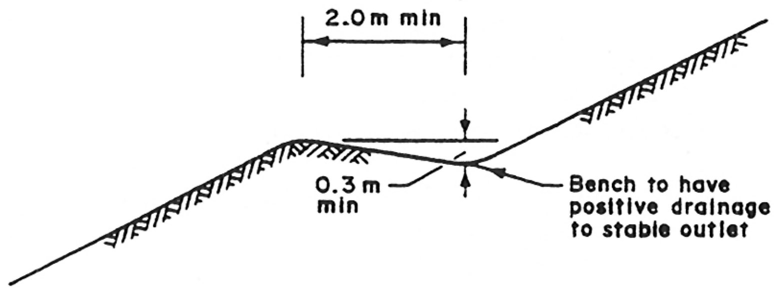
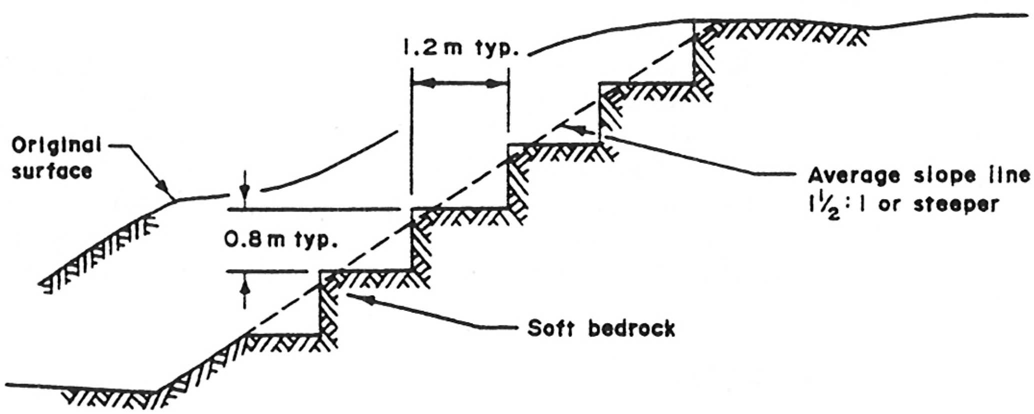


Figure 6.9: Typical Serrated Slope



Sediment Traps and Basins

Sediment Traps

The term *sediment trap* is most widely used to denote a small temporary shallow pit excavated to intercept and store sediment in minor swales and channels and at sewer inlets. The pit may be supplemented by a dike to increase its effectiveness. A trap is distinguished from a sediment basin by its much smaller size and the lower standard to which it is constructed.

Temporary traps have the following advantages over sediment basins.

- They intercept a high proportion of sediment at a low cost.
- They can be constructed at short notice, when and where needed, without a lengthy design process. For example, traps might be quickly constructed to protect a sensitive area if heavy rains were forecast during clearing and grubbing operations.
- They can easily be expanded as necessary.
- When no longer needed, or if in the way of construction, they can simply be filled in and graded over.
- They are one of the most practical forms of protection for sensitive areas during the initial construction phases or if seeding and mulching have to be delayed for a long period.
- The cost of several sediment traps close to the source of sediment is generally less than that of a large basin placed further downstream.

A potential disadvantage is that small traps may become filled with sediment during a single storm. This may be overcome by installing a silt barrier or fence downstream, or by installing a larger trap. Traps must be cleaned out when the sediment depth approaches half the total depth. A disadvantage of diked traps is that a washout during a severe storm would create worse sediment problems downstream.

Sediment traps are intended to intercept sediment from very minor storms having a return period of one year or less. They should be located to intercept concentrated flows likely to contribute sediment to sensitive areas, and should generally not drain more than 2 hectares (U.S. Dept of Ag., 1975). If the area is larger than this, consideration should be given to placing traps on smaller tributaries further upstream, otherwise a sediment basin may have to be provided. Traps may be located in ditches and swales, but reinstatement of the area after their removal is reported to be often more difficult due to the wetness of the ground. Various types of temporary trap are described below.

Temporary Sewer Inlet Trap

The simplest type of sediment trap is a shallow temporary excavation around a ditch inlet at the bottom of a grade. (Figure 6.10). Most of the sediment collects in the bottom of the excavation, allowing relatively clear water to overflow into the inlet.

Earth Outlet Traps

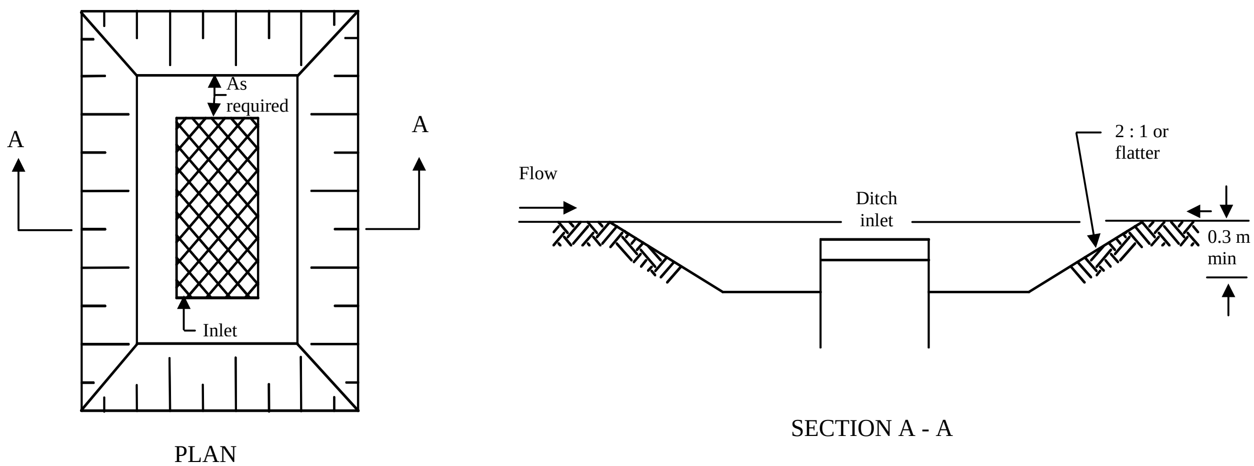
Larger traps may be located to intercept minor flows in swales or ditches (Figure 6.11(a)) or sheet flows (Figure 6.11(b)). Runoff from small storms is stored in the trap, while larger runoff flows overflow onto vegetated ground. (Figure 6.11(b)).

The excavated material may be used to construct interceptor dikes to guide the runoff to the trap, or may be used to increase the capacity of the trap. In the latter case a properly designed spillway is required. (See the section Diked Traps).

General Requirements:

- The drainage area should not exceed 2 ha (U.S. Dept of Ag., 1975).
- The location should be selected on the basis of the local drainage pattern, ease of access for clean out, and minimal interference to construction.
- The depth should normally not exceed 1.0 to 1.25 m.
- The dimensions for small traps may be based on experience, and for larger traps, dimensions may be based on 125 m^3 per hectare of drainage area (U.S. Dept of Ag., 1975).
- Side slopes should be 2:1 or flatter.
- The trap should be cleaned out when the depth of sediment approaches half the trap depth.
- After the trap has served its purpose, the area should be levelled off, compacted, seeded and mulched or otherwise finished.
- Child safety should be considered where appropriate.

Figure 6.10: Temporary Sediment Trap Around Ditch Inlet at Bottom of Grade



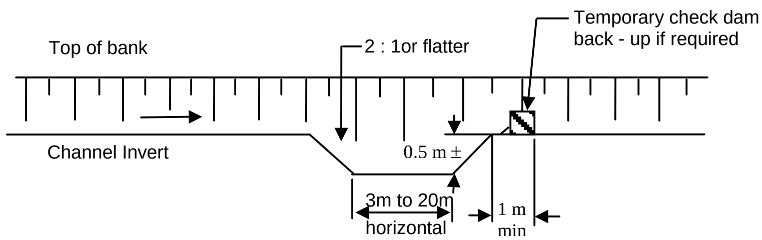
Ditch-Type Trap

A sediment trap may also be in the form of a closed-ended ditch on a zero gradient to intercept flow from an embankment before it can enter an adjacent area (Figure 6.11(c)).

Diked Trap

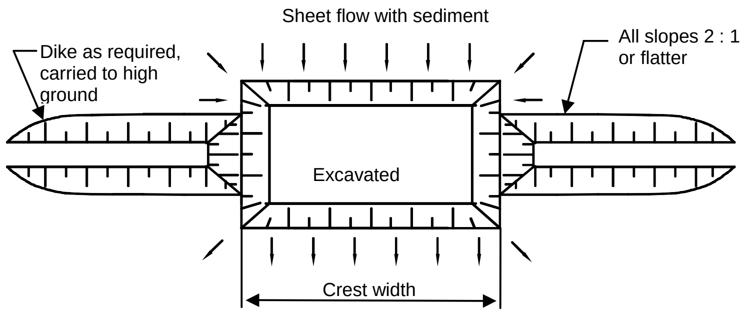
The diked sediment trap, shown in Figure 6.12 is not recommended for normal use because of the relatively costly spillway requirements. Further details are given in (U.S. Dept. of Ag., 1975) or similar publications.

Figure 6.11: Excavated Sediment Trap



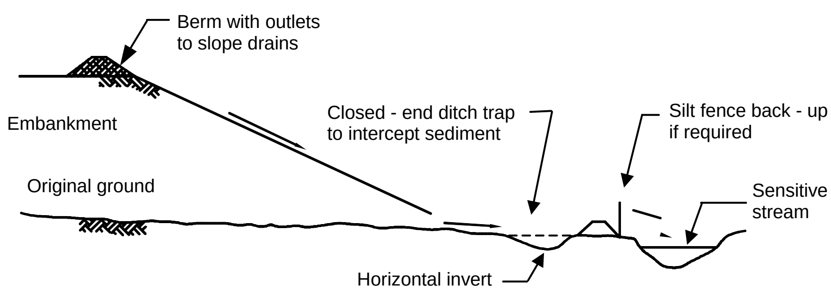
LONGITUDINAL SECTION

(a) Excavated sediment trap in channel.



PLAN

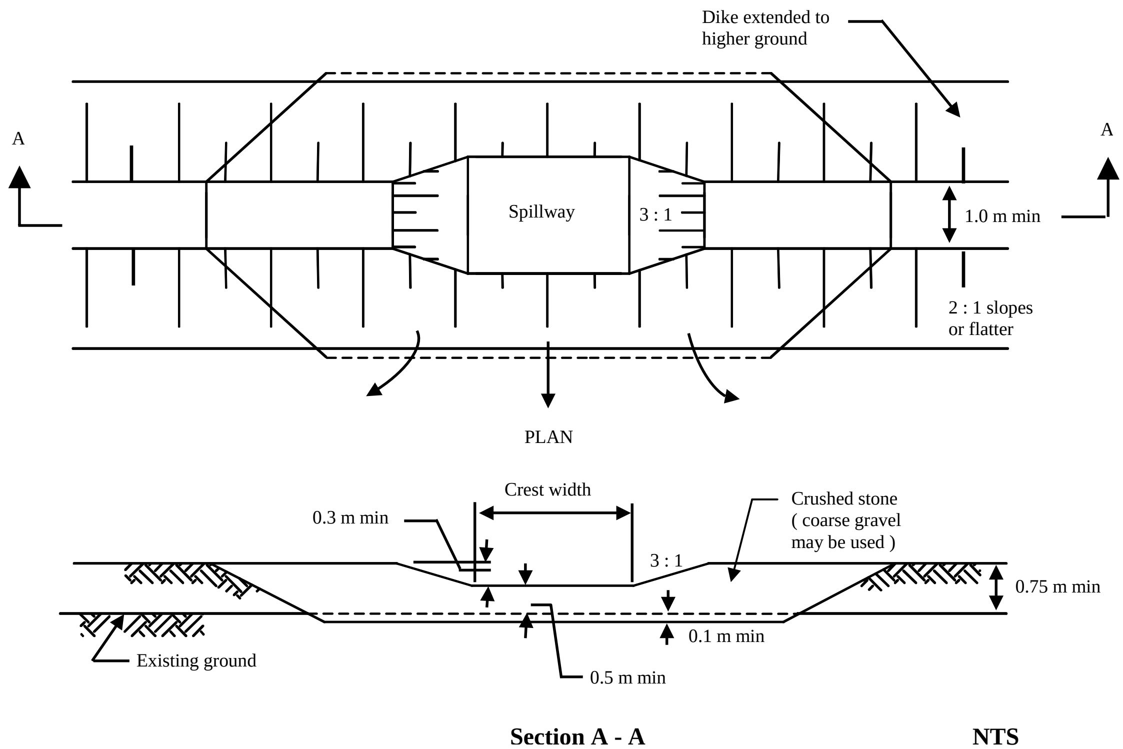
(b) Excavated sediment trap for sheet flow



CROSS - SECTION

(c) Ditch type sediment trap.

Figure 6.12: Typical Diked Sediment Trap with Stone Spillway



Sediment Basins

Sediment basins are relatively large basins for intercepting and storing sediment. They are distinguished from sediment traps by their larger size and the necessarily higher standard of design and construction. Basins may consist of an excavated or diked area, an existing depression such as an abandoned stream channel, a dam placed across a small valley, or a combination of these. Excavated, diked and natural basins are generally appropriate in relatively flat terrain, and are preferred due to their lower vulnerability to washouts. Dammed basins are less desirable because of the increased risk of failure, and the greater downstream damage potential in the event of failure.

Basins can provide an effective means of intercepting and trapping sediment before it enters an environmentally sensitive area or receiving water. Their effectiveness depends to a large extent on the size of particles entering the basin and the period over which they are detained. Coarse particles, such as sand, settle very quickly, whereas extremely fine particles may remain suspended indefinitely, as demonstrated in the Table 6.4. To provide a reasonable degree of interception, the detention period should be at least 24 hours, and for clay particles it should preferably be increased.

Sediment basins have the following potential disadvantages.

- Large sediment basins, especially those requiring a relatively high dike or dam, may be costly and may entail certain risks discussed below. They should therefore be used only where all other alternatives have been thoroughly explored.

- Improperly designed basins involve a considerable risk that in the event of a failure, damage downstream may be worse than would have occurred without a basin.
- Until the contributing drainage area has stabilized, sediment basins require cleanouts after storms to maintain their effectiveness. Consideration should be given to the requirements for the removal and disposal of sediment.
- A large area may be needed for the construction of a basin.
- Settlement of the sediment may not be complete in small basins, allowing a portion of it to pass over the spillway in large floods.

Table 6.4: Surface Area Requirements for Sediment Basins and Traps

Particle Size (mm)	Settling Velocity (m/sec.) x 10^{-3}	Surface area Required (m^2)
0.5 (coarse sand)	58	17.2
0.2 (med. sand)	20	50.0
0.1 (fine sand)	7	142.9
0.05 (coarse silt)	1.9	526.3
0.02 (medium silt)	0.29	3,448
0.01 (fine silt)	0.073	13,698
0.005(clay)	0.018	55,556

Source: Goldman et al, 1986

Surface area required, calculated assuming Q =1 m^3/s and SAAF = 1

Basin Location

- Possible locations for sediment basins include wide medians, interchange areas, abandoned channels, natural depressions, and surplus land. The location should be carefully selected to meet the following requirements.
- The basin should preferably not be on a natural watercourse, particularly a continuously flowing stream. To the extent possible, it should receive runoff only from the disturbed construction area.
- The basin should be located where it and any backed-up water will cause minimal interference to construction.
- Access should be possible for sediment removal without damage to finished areas.
- A dumping site for sediment should be available at a safe distance from the basin and any ditches or watercourses.
- Locations in sensitive wetlands should be avoided.
- Sediment basins should not be constructed at locations where a washout is likely to cause a significant danger to life or damage to property.
- The location should take account of the interests or wishes of other agencies with due consideration for added costs to MTO.

Basin Type

Sediment basins may be classified into two types (U.S. Transp. Research Board, 1980).

Type 1 basins are detention-type basins which have a principal spillway controlling the outflow for minor floods (for example, 2-year return period or less) so that it is released over a minimum of 24 hours, and an emergency spillway which accommodates larger floods. Flow over the emergency spillway will bypass the pond completely. The principal spillway may be a pipe or other conduit (Figure 6.13), or may consist of a rock fill portion of the embankment which permits filtered outflow through the rock and also serves as the emergency spillway (Figure 6.14). The drainage area for this type should not exceed 40 ha (U.S. Transp. Research Board, 1980).

Type 2 basins are retention-type basins which do not have a principal spillway, but retain the runoff from minor storms (for example, 2-year return period or less). During larger storms (for example, 5-year return period or greater) flows in excess of the pond capacity pass though the basin and exit over an emergency spillway. A type 2 basin is shown in Figure 6.15. This type of basin should be limited to drainage areas not exceeding 5 ha (U.S. Transp. Research Board, 1980). Type 2 basins should cost considerably less than Type 1, and the design is simpler.

Design of Type 1 Basin

This type detains minor floods, releasing them through a principal spillway over at least 24 hours, and allows larger floods (5-year or more) to overflow through an emergency spillway, (Figures 6.14 and 6.15). Due to their somewhat greater cost and complexity, Type 1 basins are likely to be used less frequently than Type 2. The general requirements to be considered and the design procedure of Type 2 basins are presented below. The design procedure is described in detail in Design Example 6.1. Figure 6.13 shows a typical plan and details for Type 1 basin.

General Requirements:

- The drainage area should not exceed 40 ha.
- The storage volume is measured up to a point 0.3. m below the crest of the emergency spillway to provide freeboard.
- The basin length from its inlet to the principal spillway should be at least twice its width.
- The capacity of the principal pipe spillway should be at least 0.05 m^3/s per hectare of drainage area when the water level is at the elevation of the emergency spillway. For capacity tables refer to the reference (U.S. Transp. Research Board, 1980).
- Embankment height should not exceed 3.0 m for 2:1 slopes and 4.5 m for 2.5:1 slopes.
- Special design details include.
    - a cutoff trench for the embankment core, (Chapter 4);
    - an anti-vortex device and debris rack for the riser, (Chapter 4);
    - anti-seep collars for the principal pipe spillway, (Chapter 4);

- provision for dewatering the basin, (Chapter 4);
- scour protection at the outlet of the principal spillway, (Chapter 5); and
- filter cloth for a rock fill spillway, (Chapter 5).

Design Procedure The following steps outline the design procedure.

1. Select the design particle size

The design particle size is the size of the smallest particle that will ideally settle to the bottom of the basin in the time it takes for the particle to travel the full length of a basin. Theoretically, all particles larger than or equal to the design particle size will settle out during the full length of travel in the basin. Particles smaller than the design particle size would, however, not settle out. A design particle size is chosen, based on a knowledge of the soil types in the area.

2. Estimate the particle settling velocity, V_s

Particles smaller than the design particle size have a lower settling velocity than the design particle and particles larger than the design particle size have a greater settling velocity. The settling velocities for several particle sizes are shown in Table 6.4.

3. Estimate the sedimentation basin area

The surface area of the basin can be estimated from the inflow rate, Q, and the settling velocity, V_s, based on the following relationship. The theoretical background information required for the development of this relationship is presented in Chapter 8.

$$A_s = \frac{SAAF * Q}{V_s} \qquad (6.2)$$

where:

A_s = minimum basin surface area, m^2
SAAF = surface area adjustment factor, dimensionless
V_s = settling velocity for a particle size, m/s
Q = inflow rate, m^3/s

The surface area adjustment factor, SAAF, accounts for the less than ideal conditions in the basin, where the performance of the sediment basin may be affected by the location of the outlet compared to the inlet (short circuiting) turbulence, wind and poor design. Figure 6.16 provides the relationship between the area adjustment factor, turbulence and the percentage of sediment basin efficiency (Goldman et al, 1986). It should be noted that the surface area is independent of basin depth or volume.

To determine the length and width of the basin a length to width ratio of 2:1 can be assumed initially. If a longer facility is required, as a result of the analysis of the settling depth or due to site specific constraints, the length to width ratio may be adjusted.

Figure 6.13: Typical Type 1 Sediment Basin with Principal Pipe Spillway

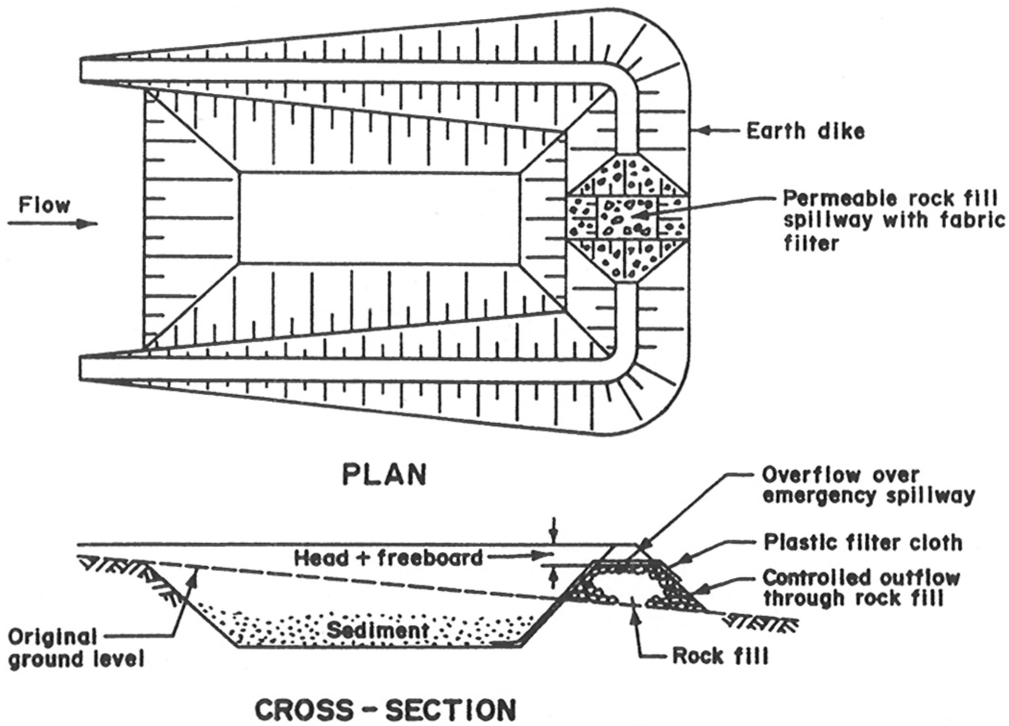


Figure 6.14: Typical Emergency Spillway

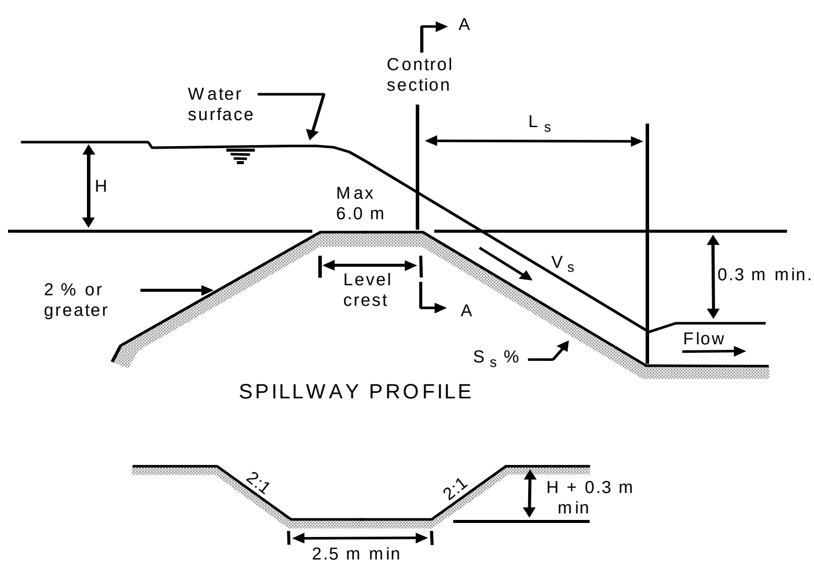


Figure 6.15: Typical Type 2 Sedimentation Basin

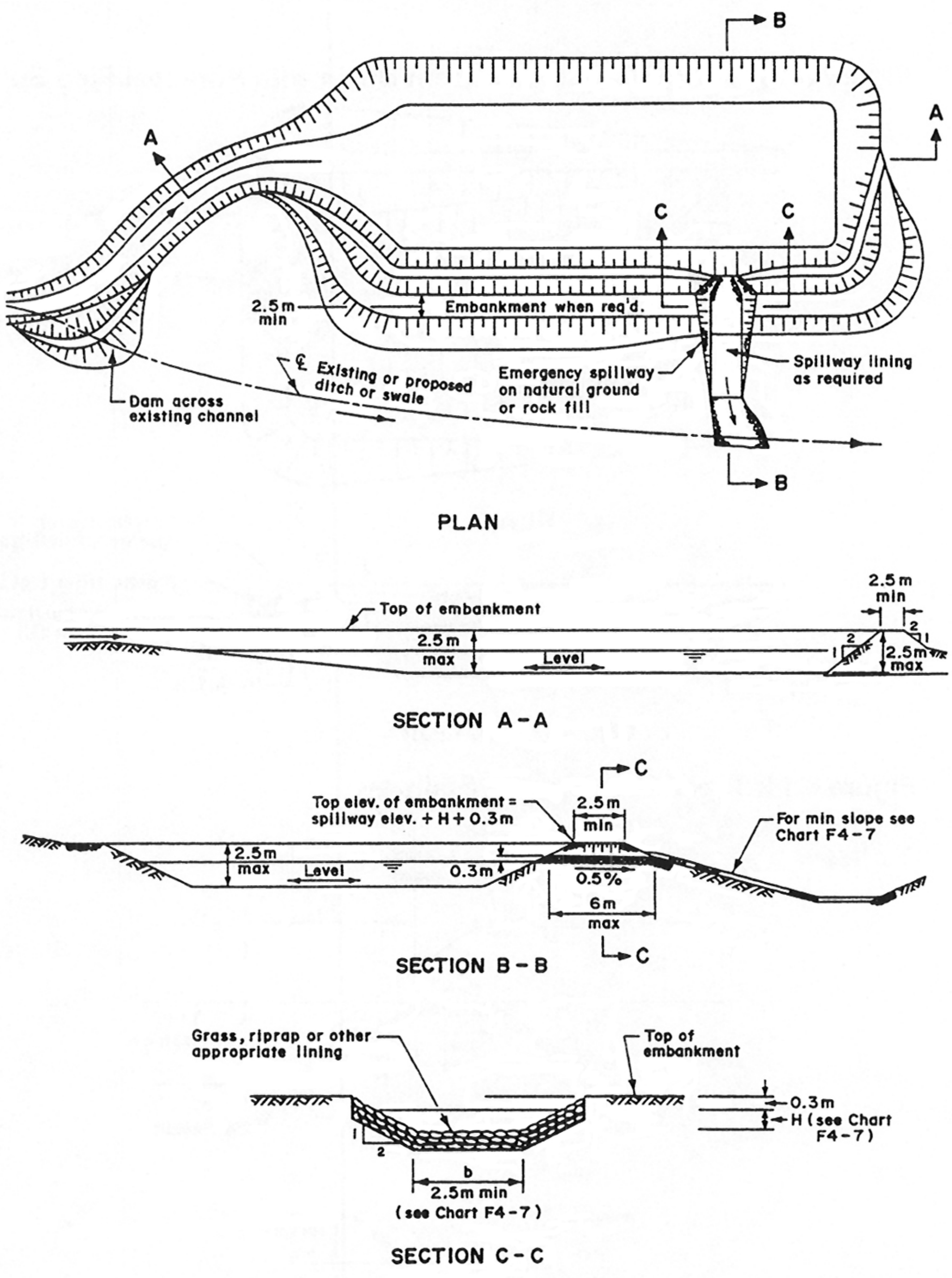
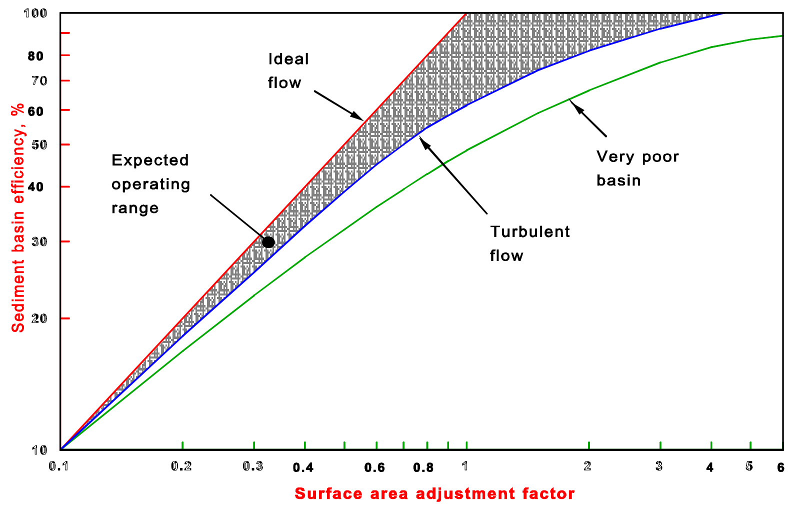


Figure 6.16: Effect of Turbulence on Surface Area Adjustment Factor



Goldman et at (1986)

4. Estimate the settling depth

The sediment basin must have a sufficient cross-section area such that the horizontal component of the flow-through velocity, V_h, is less than the velocity that can re-suspend (or scour) particles already settled in this basin. The velocity that causes settled particles to be re-suspended is termed the *scour velocity*, V_{sc}.Therefore,

$$V_h \leq V_{sc} \tag{6.3}$$

The scour velocity is given by,

$$Vsc = \frac{R^{1/6}}{n}\left[k(\gamma - 1)\frac{d}{1000}\right]^{1/2} \tag{6.4}$$

where:

R = Hydraulic radius of basin cross-section, m
n = Manning's roughness coefficient for basin lining
k = shape coefficient (0.04 for granular material)
γ = specific gravity of the sediment
d = particle size, mm

The horizontal velocity is given by;

$$V_h = Q / (W * H_{set}) \tag{6.5}$$

where:

Q	=	inflow rate, m^3/s
W	=	basin width, m
H_{set}	=	settling depth, m

Having estimated the basin width, W, in the previous step, the depth of the basin becomes the controlling factor in limiting resuspension of sediment.

5. Estimate soil loss and sediment delivery

Soil loss from an area contributing to a settling basin needs to be determined in order to assess the depth of the soil storage portion of the basin. Soil loss can be determined using mathematical methods such as the Universal Soil Loss Equation (USLE) developed by the Agricultural Research Service, or more accurately, the Modified Universal Soil Loss Equation (MUSLE) (Williams, 1975). These methods predict soil loss caused by sheet and rill erosion during highway construction. A comprehensive discussion on the theory behind these methods is provided in Chapter 8. The application of the MUSLE is covered in Design Example 6.1.

Sediment delivery is the sediment produced by erosion that reaches the target area. Much of the sediment travels only a short distance before it is deposited. The amount of sediment reaching a sensitive area may, therefore, be significantly lower than the estimated soil loss.

The ratio of sediment delivered to a given point to the quantity eroded is termed the sediment delivery ratio, SDR. The SDR closely approaches 1.0 if the point of delivery is immediately downstream from the eroding area, but becomes very small after the flow passes through a substantial buffer of dense vegetation or forest litter.

The ratio of the sediment delivered at a given location to the gross soil loss may be written as,

$$Y = E * SDR \tag{8.97}$$

Design Chart 6.5 gives the SDR as a function of travel surface and distance. Refer to Chapter 8 for further discussion.

6. Estimate the sediment storage depth, H_{STOR}

The storage depth for sediment deposition must be sufficient to contain all the sediment reaching the basin. The storage depth may be estimated from;

$$H_{STOR} = Y * A_s \tag{6.6}$$

where:

A_s = Surface area of the sediment storage portion of the basin.

7. Estimate of the total basin depth, H_{TOTAL}

The total basin depth consists of storage depth, H_{STOR}, settling depth, H_{SET}, and freeboard.

$$H_{TOTAL} = H_{STOR} + H_{SET} + \text{Freeboard} \quad (6.7)$$

It is important to ensure that there is adequate volume in the basin, above the storage depth, to store the runoff from a frequent storm (2-5 year storm). The runoff volume may be estimated using the relation:

$$V = 0.476 * Q_p * t_p \quad (8.94)$$

where:

V = runoff volume, (ha-m) = storage volume
Q_p = peak inflow rate, m^3/s
t_p = time to peak, hours = $0.7\ t_c$
t_c = time of concentration (Bransby-Williams formula) (Design Chart 1.11). Refer to Chapter 8 for the details.

Design of Type 2 Basins

With this type of basin the whole of a 2-year flood (or less) is stored and larger floods flows over the emergency spillway (Figure 6.15). A simple design procedure is given below. Detailed analysis procedure and further refinements may be obtained from the publication *Design of Sedimentation Basins and Implementation Procedures* (U.S. Transportation Research Board, 1980).

Design Procedure The following steps outline the design procedure.

1. Calculation of the required volume

Based on a water quantity of 125 m^3/ha, the dimensions necessary to give the required storage volume up to an elevation 0.3 m below the spillway elevation can be calculated. The depth may be attained by excavation and/or an embankment. The ratio of width to length is not important for a Type 2 basin because the entire design inflow volume (2-year or less) is retained in the basin.

2. Emergency spillway

The design of the emergency spillway is part of the design procedure common to Type 1 and Type

2 basins. In the case of Type 1 basins, blockage of the main spillway may occur and, therefore, the design of the emergency spillway should not account for any flow through the main spillway. The details of the hydraulic analysis of embankment spillways is provided in Chapter 8.

The design procedure of an emergency spillway is as follows:

1. The peak flow rate, Q_p, for the design storm can be determined by the Rational method (Chapter 8).
2. If the spillway lining is to be other than grass (n = 0.04), Q should be adjusted, as shown in Design Chart 6.6, Sheet 2.
3. A suitable combination of head and spillway width is selected (minimum width 2.5 m normal to flow) from Design Chart 6.6, Sheet 1.
4. The spillway outlet slope, S_s, length, LS, and velocity, V_s, are determined from Design Chart 6.6, Sheet 1. For adjustments to V_s, refer to. Design Chart 6.6, Sheet 2.
5. If the velocity exceeds the allowable velocity for grass (1.8 m/s or other appropriate value) the designer should decide on appropriate erosion control measures requirements .
6. The spillway location should be selected such that it will be on natural ground. Alternatively it can be constructed on rock fill or provide riprap.

3. Determine embankment details

- Height = spillway height + head (from Step 2.3 above) + 0.3 m freeboard. (Maximum height = 2.5 m)
- Top width = 2.5 m minimum.
- Side slopes = 2:1 or flatter.
- Material to be impervious and well-compacted.

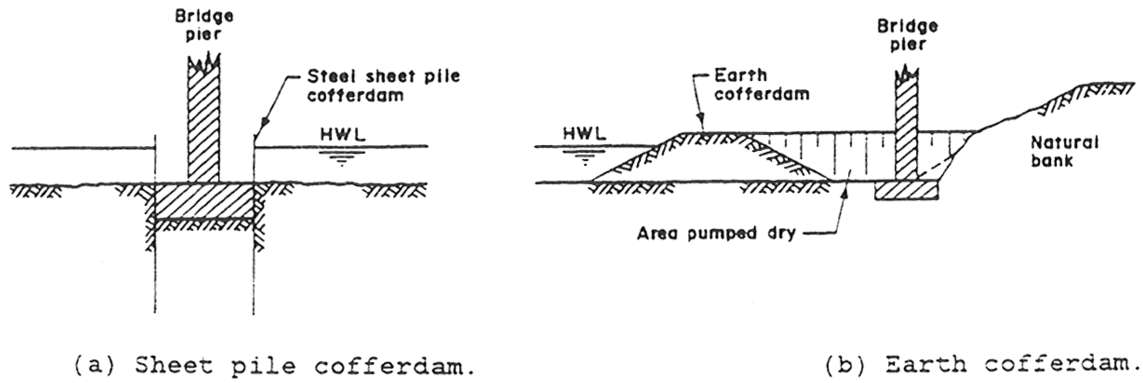
Construction in the Dry

Sediment problems can often be minimized at construction sites in lakes, streams, and wetlands by carrying out the work in the dry. This may be achieved by constructing a cofferdam or, in the case of a stream, a temporary stream diversion. As with other types of control, the cost must be commensurate with the environmental and engineering benefits.

Cofferdams

A cofferdam is a temporary dam constructed of earth, sheet piling or other material to enclose a work area and permit the removal of water (Figure 6.17). Some types of cofferdam, including sheet piles, may be very costly, especially in deep water. in these situations, other modes of construction may have to be adopted.

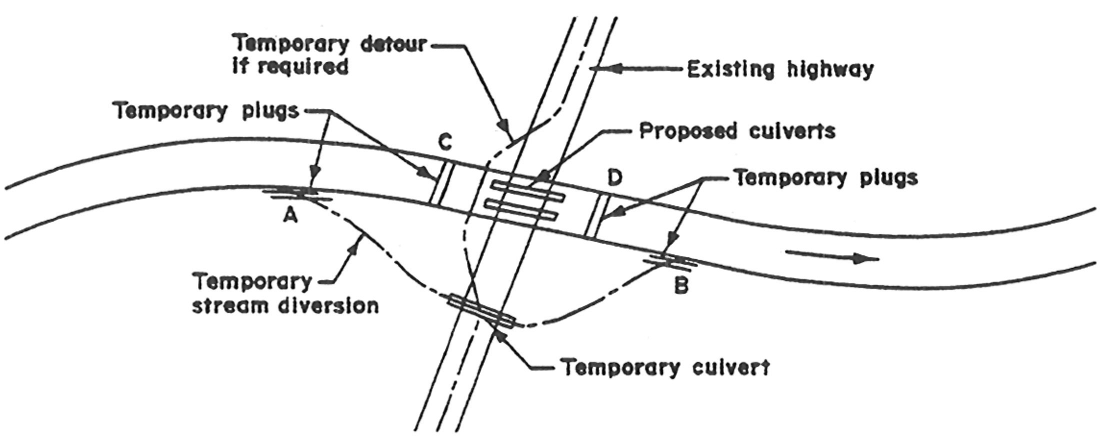
Figure 6.17: Typical Cofferdams



Temporary Stream Diversions

Temporary stream diversions (Figure 6.18) may be provided to minimize sedimentation resulting from the construction in and around environmentally sensitive streams. The decision of whether or not to divert a stream should normally be considered as part of the total design approach in the preliminary design stage.

Figure 6.18: Temporary Stream Diversion for Construction of Culverts



A possible disadvantage of earth cofferdams is that erosion of the outer slopes may contribute sediment to the surrounding waters.

Some considerations in the use of cofferdams are as follows.

- The width of a cofferdam relative to a stream channel should not be large enough to produce unacceptable scour or velocities.
- The material used for earth cofferdams in environmentally sensitive waters should be clean granular without a significant content of silt or clay. However, it should be noted that such material may be excessively permeable under any significant head of water.
- Cofferdams should not present an undue hindrance to the passage of fish and boats.
- Cofferdams should be removed as carefully as possible to minimize sedimentation.

Portable fabric cofferdams have been used in some countries for several years, and may merit consideration for suitable sites in Ontario. They have the advantage that their installation minimizes turbidity. They consist of an L-shaped tubular steel frame with plastic-coated synthetic woven material attached to the top by fabric loops. Sections of fabric are joined by Velcro-type fasteners. Once the fabric is in place, water pressure seals the interior and the area can be de-watered. The portable dam is best used in water less than 3 m deep with a firm bottom (Tech. Rep. DS-78-13, U.S. Army Engr., 1978).

Sediment Control in De-watering Operations

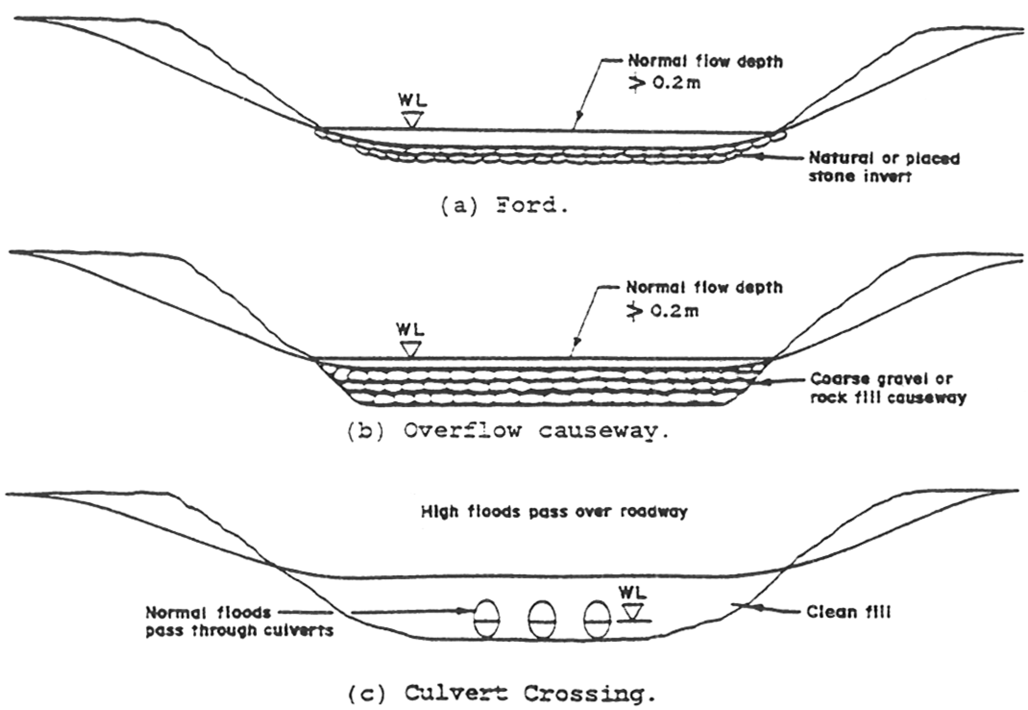
Before passing water pumped from excavations into lakes, streams, wetlands or other sensitive areas, excessive sediment should be removed by means of silt traps, filters, sediment barriers or heavily vegetated buffer strips at least 10 m wide in the direction of flow.

Temporary Stream Crossings

Temporary crossings (Figure, 6.19) may be needed for construction equipment or road traffic, or both. The requirements are similar in both cases except that the design standards will normally be lower for construction crossings. Similar to stream diversions, the decision to provide a temporary stream crossing should be considered early as part of the preliminary design process. The following conditions should be observed where sensitive streams are involved.

- The number and locations of construction crossing on a project should be closely controlled.
- Fish passage on migration routes should not be hindered by the crossing during the migration period .
- Temporary crossings should not be constructed or removed during periods of spawning or significant fish runs.
- Materials used for constructing fords, causeways or temporary road embankments should be coarse gravel or stone not containing significant amounts of silt or clay.
- Temporary bridges and culverts should be large enough to pass minor floods without damage or erosion. Wherever possible, larger floods should be allowed to pass over the temporary roadway. The crossings should not cause flooding of buildings or other valuable property. (See Chapter 5).
- Temporary crossings should be removed when they are no longer needed, and the stream bed and banks restored to their original condition or better.

Figure 6.19: Typical Temporary Construction Crossing



Sediment Barriers and Filters

Sediment barriers or filter are relatively inexpensive devices in the form of permeable or semi-permeable fences or dams. They may consist of filter fabric, burlap, straw bales, gravel berms, brush piles and many other materials. Sediment barriers are primarily intended for reducing the velocity of sheet flow and thereby inducing the deposition of sediment, which is retained by the barrier while the water passes through. They are simple to construct when and where needed, and are especially valuable in the early stages of construction.

Barriers should be installed before earthmoving operations commence. They should be located where they can intercept sediment before entering a sensitive environment. Typical locations are:

- along the toe of an embankment slope;
- along the top of bank of a sensitive stream;
- along the edge of the right-of-way to protect sensitive areas;
- around storm sewer inlets;
- downstream from slope chutes; and
- as a back-up downstream from a sediment trap.

General Requirements:

- Sediment barriers should normally be required to intercept sediment only from small sheet flows on relatively flat gradients. This is necessary because the barriers are structurally weak, and are therefore easily damaged or destroyed by flows of any significant magnitude.
- The strength and effectiveness of barriers intercepting sheet flows may be increased by constructing them in the shape of a horseshoe with its ends pointing upstream.
- Water and sediment must be prevented from passing underneath the barrier by recessing the barrier into a trench and carefully compacting the backfill.
- Sediment barriers must be adequately maintained and repaired, especially after storms. It should be remembered that the failure of a sediment barrier may allow a considerable quantity of accumulated sediment to pass downstream. To avoid this, sediment must be cleaned out whenever its depth approaches half the height of the barrier, and the barrier must be inspected after every rainfall so that weak points are repaired as soon as they develop.
- After removal of a sediment barrier, any sediment left in place should be graded, seeded and mulched or otherwise finished.

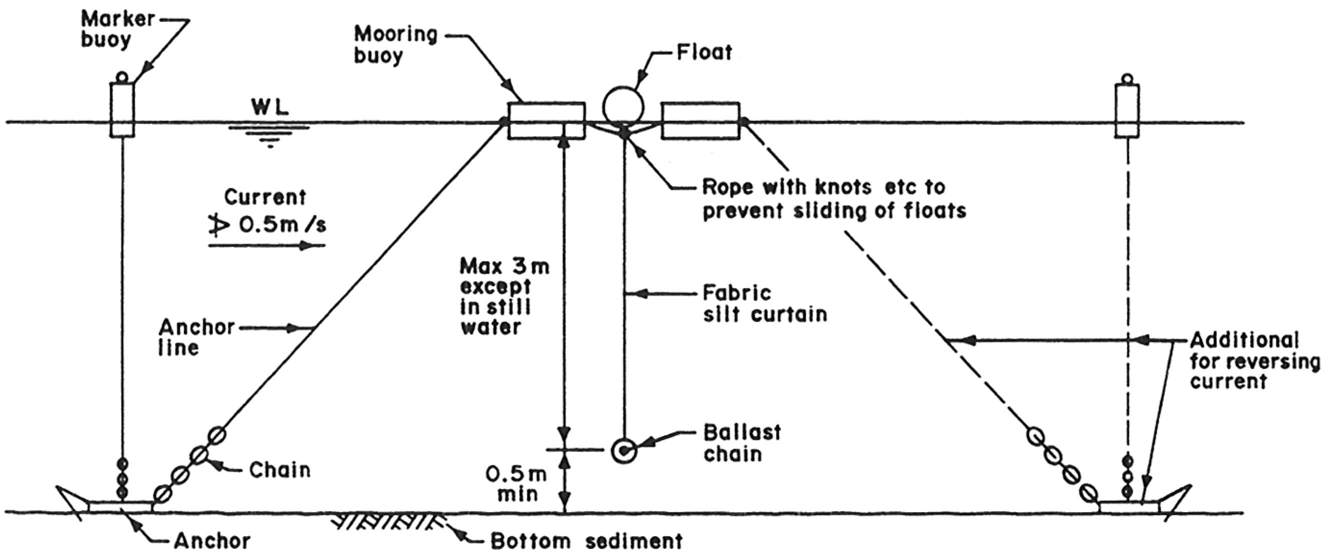
Silt fences are the most effective barriers for sheet flows, followed by burlap barriers (for limited periods) and straw bale barriers (Sherwood and Wyant, 1979). These types of control measures are not discussed in this chapter. This information is in other Ministry of Transportation manuals, such as the *Environmental Manual, Erosion and Sedimentation Control*, working draft (MTO, 1994).

Solid or semi-solid sediment barriers (sandbags or straw bales) constructed in ditches and swales are considered to be temporary check dams, and have been mentioned earlier in this chapter.

Silt Curtains

Suspended sediment produced by construction in or near a slowly flowing river, lake or wetland may sometimes be contained by a filter fabric (geotextile) barrier. In shallow lakes this may be mounted on posts driven into the bed. In deeper water a curtain may be suspended from ropes supported by floats and suitably anchored (Figure 6.20). Silt curtains, in general, may be very costly and highly vulnerable to damage or destruction by strong winds, waves, current, ice and boats.

Figure 6.20: Typical Silt Curtain Anchorage System Using Marine Anchors



The following point should be considered to ensure that silt-curtains are used only where there is ample justification and a reasonable chance of survival. The considerations are based largely on research by the (U.S. Corps of Engineers, 1978), with additional information from an internal report by MTO's Eastern Region (Jones, 1983).

Considerations for the Use of Curtains

The decision whether or not to provide a silt curtain at a given site should include consideration of the nature of:

- the disturbed soil and sediment, recalling that only the finest sediment will remain in suspension for any significant time or travel for any distance;
- the nature of the construction operation (dredging, excavation, fill dumping etc.);
- the natural sediment load of the lake or stream during a full range of conditions (i.e. strong

winds, spring floods, waves, etc.);
- the significance of impacts on the surrounding environment if no containment was provided; and
- the absence of suitable alternatives.
- silt curtains should not be used at locations where they will be exposed to:
  - currents stronger than 0.5 m/s;
  - high winds;
  - large or breaking waves;
  - impact by ice flow; and
  - prolonged freezing temperatures sufficient to create an ice sheet.
  - passage of boats

Further consideration should be given to the following:

- Silt curtains should be aligned obliquely to the current or prevailing wind.
- Curtains should be suspended by a rope and float system except for small barriers where there is no current. The floats should be capable of supporting 5 times the weight of curtain, and should be prevented from sliding along the rope by knots, clamps or other devices. The bottom edge of the curtain should be weighed down by a ballast chain. (See Figure 6.20).
- The depth of a curtain should be such that the bottom edge will not be lower than 0.5 m above the lake or stream bed. This is necessary to prevent the curtain from being pulled down by the weight of accumulated sediment and to facilitate salvaging the curtain.
- If there is likely to be a current, the curtain should not extend to more than 3 m from the surface, to prevent it from billowing up due to the force of the current.
- Adequate anchorage is vitally important for the success of a silt curtain. The landward end should be secured to a pile, a post anchored to a deadman, or another immovable object. The remainder of the curtain may be secured to piles, where piles can be driven into the lake or streams bed, or marine anchors. For difficult or costly installations of marine anchors, a marine expert should be consulted.
- It is essential to use fabric which is strong enough for the intended purpose. Seams in the curtain fabric should be thermally welded rather than stitched, to prevent tearing.

Sediment Removal and Disposal

Timely removal of sediment is essential for the continued effectiveness of sediment control measures. It is generally accepted that the material should be removed before its depth exceeds half the height or depth of the barrier, trap or basin, and also immediately after moderate to severe rainstorms. To facilitate cleanout and minimize damage to finished areas, convenient access should be provided for equipment.

Disposal of sediment should be given careful consideration. The material should not be dumped adjacent to the barrier, trap or basin or anywhere where it could contribute sediment to a sensitive environment. The sediment should be spread and graded and the area finished to the same standards as the surrounding ground.

Precipitation of Suspended Sediment

The precipitation of colloidal sediment in water can be difficult and costly, and should be undertaken only in extremely unusual and rare circumstances. Various types of flocculants, filters, coagulants and settling ponds may be used for this purpose. Care should be taken that the chemicals do not themselves pollute the water.

Design Example 6.1
Sediment Basin

Required

A construction site is located in an area characterised by relatively erosive silty clay loam soils. A Type 1 sedimentation basin is required to trap the sediment flow from this site.

Given

- The drainage area at this site is 10 ha.
- The 5-year peak discharge, $Q_5 = 0.88\ m^3/s$
- Soil characteristics:
  - shape coefficient, k= 0.04
  - specific gravity, γ = 1.29 (submerged clay-silt-sand mixture, SCS, 1969)
  - design particle dia., D = 0.05 mm
  - Manning's coeff., n = 0.018 (unlined earth channel) (Design Chart 2.01)

Analysis

1. Select a design particle size

Design particle size, D = 0.05 mm (given)

2. Estimate the particle settling velocity, V_s

V_s = 0.0019 m/s

where :

d = 0.05 mm (Table 6.4)

3. Estimate the sedimentation basin area, A_s:

$$A_s = SAAF * Q / V_s \qquad (6.2)$$

where:

Surface Area Adjustment Factor, SAAF	= 1.2	(Figure 6.16)
Design Flow, Q	= 0.88 m^3/s	
Particle Settling Velocity, V_s	= 0.0019 m/s	

$$A_s = 1.2 * \frac{0.88\ m^3/s}{0.0019\ m/s} = 556\ m^2$$

With a desirable length to width ratio of 2:1 and a surface area of 556 m^2, the dimensions of the basin, rounded to the nearest metre, are: L_b = 33 m and W_b = 17 m.

4. Estimate the settling depth

From the following equation (Goldman et al, 1986), the scour velocity is given by:

$$V_{sc} = \frac{R^{1/6}}{n} \left[k\,(\gamma - 1) \quad \frac{d}{1000} \right]^{1/2} \qquad (6.4)$$

where:

R = hydraulic radius, m (to be determined)
k = 0.04 (granular material)
γ = 1.28 (submerged clay-silt-sand mixtures, SCS, 1969)
D = 0.05 mm
n = 0.018 (unlined earth channel) (Design Chart 2.01)

$$V_{sc} = \frac{R^{1/6}}{0.018} \left[0.04 \quad (1.28 - 1) \quad \frac{0.05 \quad mm}{1000} \right]^{1/2} \qquad (6.5)$$

V_{sc} = 0.0416 $R^{1/6}$

The horizontal component of inflow velocity is given by the following equation:

V_h = Q / (W * H_{set}) (6.5)

The horizontal component of inflow velocity in the basin, V_h , must not exceed the scour velocity, V_{sc}, to prevent settled particles from being re-suspended, that is, $V_h \leq V_{sc}$

Substituting for V_{sc} and V_h,

0.0416 $R^{1/6}$ ≥ Q / (W * H_{set}) (6.8)

Since the settling velocity, V_s, is defined by

V_s = Q / (W * L_b)

For V_s = 0.0019 m/s, the above equation becomes,

0.0019 = Q / (W * L_b)

Rearranging this expression gives:

$$\frac{Q}{W} = 0.0019\ L_b$$

Substituting into Equation (6.8),

$0.0416 * R^{1/6} \geq 0.0019 * L_b / H_{set}$

gives the relationship,

$21.88 * R^{1/6} \geq L_b / H_{set}$

or: $L_b \leq 21.88 * R^{1/6} * H_{set}$ (6.9)

The ratio of basin length, L_b, to settling depth, H_{set}, should be kept below this value. Assuming

$H_{set} = 1.5$ m, therefore,

$L_b / H_{set} = 33$ m/1.5 m = 22

For a trapezoidal section of bottom width 15 m and 2.5:1 side slopes, the cross-sectional area, A_b, wetted perimeter, P, and hydraulic radius, R, are given by:

A_b = (15 m + 2.5 H_{set}) * (H_{set})
= (15 m + 2.5 * 1.5 m) * (1.5 m)
= 28.13 m^2

P = Install Equation Editor and doubl click here to view equation.
= 23.1 m

R = A_b / P = 1.22 m

and, $21.88\ R^{1/6}$ = 22.6

The above value of the ratio L_b/H_{set} (= 22.6) is greater than the minimum value (= 22) and therefore the settling depth of 1.5 m is satisfactory.

5. Soil loss and sediment delivery

The soil loss from a construction site with 10 ha contributing area in Chatham District, North of Dresden, is estimated as 1590.4 tonnes for 5-year discharge of 0.88 m^3/s for silty clay loam soils with erodibility (K) of 0.32. This was estimated using the MUSLE and Design Charts 6.01 to 6.04.

The ratio of the sediment delivered at a given location to the gross soil loss is defined as the Sediment Delivery Ratio (SDR). Assuming a sheet flow travel distance of approximately 80 m and using Design Chart 6.05 gives a SDR of 0.4. Accordingly, the quantity of sediment reaching the basin is:

$$Y = E * SDR$$
$$= 1590.4 * 0.4 \qquad = 636 \text{ tonnes}$$

Assuming specific weight, γ, for the uncompacted sediment of 1.28 tonnes/m^3 (given), a storage volume of (636 tonnes / 1.28 tonnes/m^3 =) 497 m^3 is required.

6. Estimate the sediment storage depth, H_{STOR}

The storage depth for sediment deposition must be sufficient to contain all the sediment. The storage depth may now be estimated from,

$$H_{STOR} = Y / (\gamma * A_s)$$
$$= 636 \text{ tonnes} / (1.28 \text{ tonnes/m}^3 * 556 \text{ m}^2)$$

$$H_{STOR} = 0.9 \text{ m.}$$

7. Estimate the total basin depth, H_{TOTAL}

The total basin depth consists of storage depth, settling depth and freeboard. Freeboard of 0.3 m will be provided above the storage and settling depths.

Substituting values for storage depth, settling depth and freeboard into the following equation,

$$H_{TOTAL} = H_{STOR} + H_{SET} + \text{Freeboard}$$
$$= 0.9 \text{ m} + 1.5 \text{ m} + 0.3 \text{ m}$$
$$= 2.7 \text{ m}$$

8. Determine the runoff volume, V

For a peak inflow rate, Q_p, = 0.88 m^3/s and a time to peak, t_p = 0.19 hours, the runoff volume reaching the pond is:

$$V = 0.476 * Q_p * t_p \qquad (8.94)$$
$$= 0.476 * 0.88 * 0.19$$
$$= 0.0796 \text{ ha-m}$$
$$= 796 \text{ m}^3.$$

The volume available in the basin above the sediment storage depth for a trapezoidal section, with

2.5:1 side slopes, a depth of H_{SET} = 1.5 m and basin length L_b = 33 m. Based on these values, the volume is 1145 m^3. There is sufficient storage volume available in the basin.

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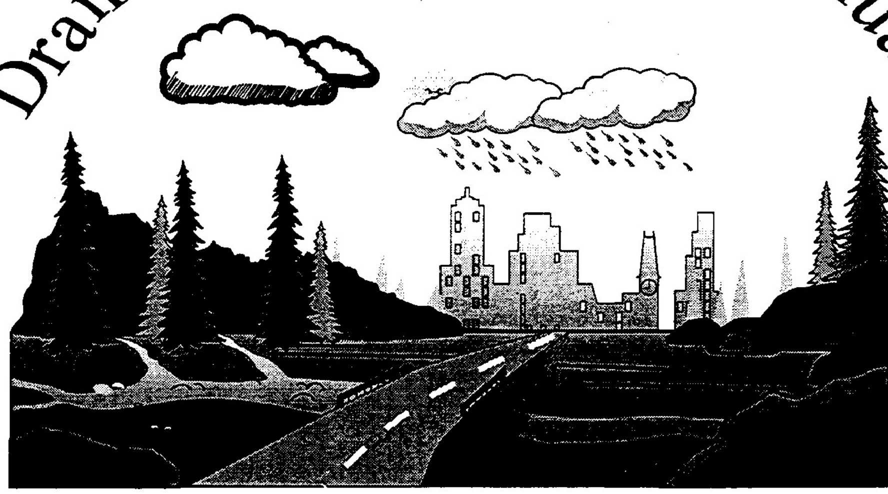
Ministry of Transportation
Ontario

MTO
Drainage Management Manual



Chapter 7
Data Sources and Field Investigations

Drainage and Hydrology Section
Transportation Engineering Branch
Quality and Standards Division

7 Table of Contents

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Purpose of This Chapter

This chapter is written primarily for engineers and technicians. It is intended to provide them with a quick reference of information useful for undertaking data collection or field investigation for drainage management planning and design for a highway project.

This chapter consists of two main sections: Primary Data Sources and Field Investigations. The section Primary Data Sources provides lists of primary sources of data most often used in hydrologic and hydraulic studies. A commentary accompanies these lists to highlight points of special interest of the data. This section does not specify what data is needed for a project as data needs of a project are covered in the individual chapters of this manual where design subjects are discussed. Nor is this section a substitute for the advice and services of data providers such as surveyors, photogrammetric/ remote sensing specialists, climatologists and biologists.

The section Field Investigations deals with hydrologic and hydraulic investigations. It is a first-cut list of what information a field investigation can gather and what things should come to mind of the investigation team when organizing a field investigation. This section is not a substitute for an operational plan and procedure for a field investigation.

Primary Data Sources

The sources of primary data outside of the ministry available for drainage management planning and design are listed in Tables 7A.1 and 7A.2 in Appendix 7A. Table 7A.1 is a more general list while Table 7A.2 singles out data in map form for mention. These two tables are not an exhaustive listing of all sources of data. For example, they have not included sources of data collected by various parties for special purposes or individual projects.

The types of data listed in Tables 7A.1 and 7A.2 include:

- Topographic Data.
- Land Use and Ground Cover Data.
- Airphotos.
- Soil Data.
- Precipitation and Other Climatic Data.
- Stream Flow Data.
- Stream Water Quality Data.
- Lake and Harbour Water Levels.
- Fish, Wildlife and Terrestrial Data.

In the two tables, notes are included to describe the nature of the data briefly. Where additional information on certain types of data is considered useful, the additional information is provided in the Commentary below.

Commentary

Topographic Maps

Ontario Base Maps (OBM) and maps of the National Topographic Series are the major sources of topographic data. These maps are available in a variety of scales. OBMs have scales ranging from 1:2,000 to 1:20,000 and National Topographic Series Maps have scales ranging from 1:25,000 to 1:2,000,000. A 1:10,000 OBM has 5-m contour intervals and a 1:2,000 OBM 2-m.

The level of detail contained in a map depends on the scale of the map. The contrasts can be seen in the maps in Figures 7.1 to 7.3. Figure 7.1 shows part of a 1:50,000 scale National Topographic Map. The portion shown in the figure covers an approximate area of 50 km^2. Figure 7.2 is a 1:10,000 Ontario Base Map and covers an area of approximately 2 km^2. Figure 7.3 is a 1:2,000 Ontario Base Map and covers an area of 8 ha.

Many municipal engineering departments have large scale topographic maps with scales of 1:1000 and 1:500. These maps often have 2-m contour and provide useful information for detail design, such as river meanders, roadside ditches, streets and layout plans of utilities.

Figure 7.1: 1:50,000 Topographic Map

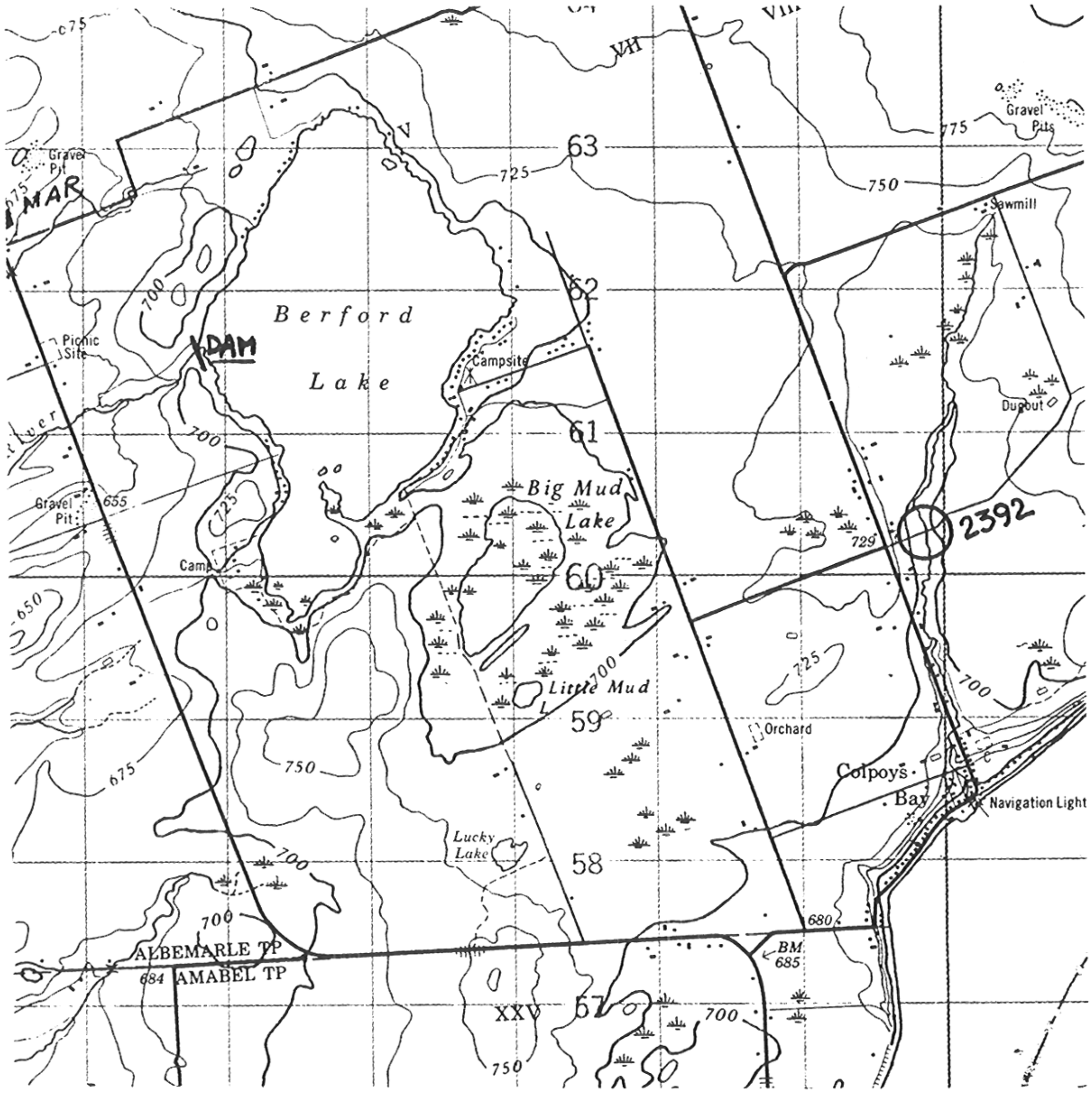
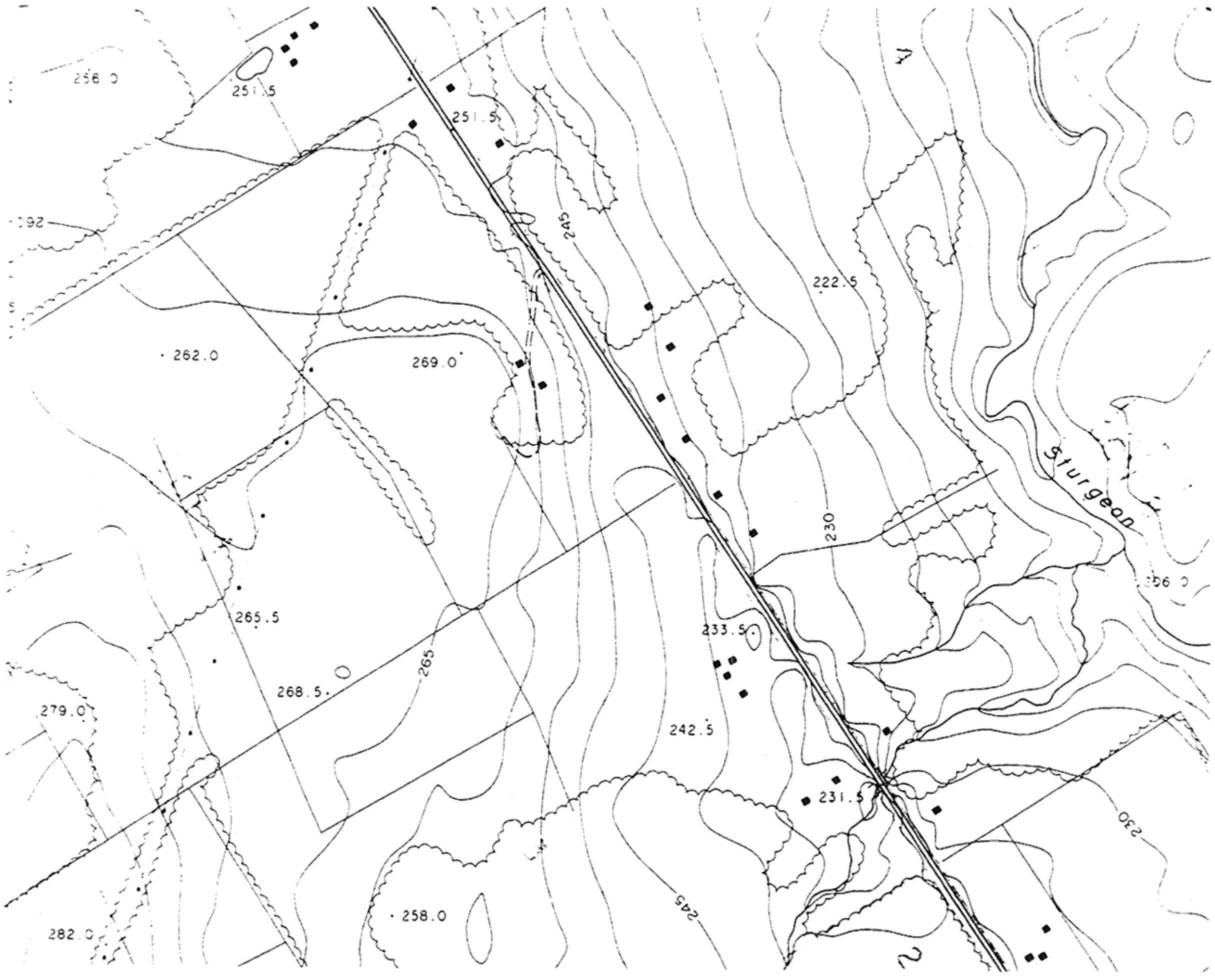


Figure 7.2: 1:10,000 Ontario Base Map



Air Photographs

A variety types of data such as topography, land use, soil and vegetative cover can be obtained from airphotos by photogrammetric and/or remote sensing methods. Mosaics are assemblies of airphotos which provide a wider coverage.

At a scale of 1:20,000, airphotos may contain a broad level of detail, such as:

- Overall terrain pattern, direction of ground slopes and surface drainage patterns.
- Layout of wetlands, marshes, lakes, rivers, forest stands, etc.
- Layout of highways and roads.

At a scale of 1:3,000, airphotos may contain details such as:

- Layout of bridges, culverts and roadside ditches.
- Position of inlet grates and catchbasins.
- Types of paved roadway, buildings and other ground features.

The accuracy of photogrammetric and remote sensing surveys depends on the quality and scale of airphotos; method and instrument used in data extraction; and human factors in operators.

The best time to take airphotos is immediately after snowmelt when soils are saturated, water bodies are full and no leaves are on the trees to obscure the ground.

Figure 7.3: 1:2,000 Ontario Base Map

Precipitation and Other Climatic Data

Atmospheric Environment Services (AES) of Environment Canada is the primary agency to gather precipitation and other climatic data. The data is supplied to requisitioners in printed pages and digital files. The *logical* formats of the data supplied in these two storage media are the same.

Precipitation Data

The raw data is usually recorded at 5-minute intervals and synthesized by AES into various data groups. Among them are:

- Daily records of hourly data.
- Monthly records of daily data
- Annual records of monthly data.

Daily data may be reported in different time intervals. The most common intervals are 5, 10, 15 and 30 minutes and 1, 2, 6, 12 and 24 hours.

Table 7.1 shows a sample of AES precipitation data in the printed form.

When using precipitation data collected by parties other than AES, it is important to note whether the data collection and synthesis follow AES's quality control and standards of equipment and procedures.

Temperature Data

Temperature data is reported in daily maximum, minimum and mean; monthly maximum, minimum and mean; and so forth.

Stream Flow Data

Water Survey of Canada is the primary agency to collect stream flow data in cooperation with conservation authorities. Stream flow data is typically reported as discharges: daily mean, monthly total, monthly mean, maximum daily, minimum daily, annual total, annual mean, maximum daily of the year and minimum daily of the year. Occasionally, maximum instantaneous discharges are also reported for a few gauging stations. Table 7.2 shows a sample of stream flow records. Some gauging stations also record sediment loads. Table 7.3 shows a sample of sediment load records.

In interpretation of stream flow data, it is important to note the instrumentation and procedure used in data collection and whether the flow is regulated by structures such as dams and weirs and whether the flow downstream of the gauging station is subject to tailwater effects.

Stream Water Quality Data

In Ontario, the primary agency to collect stream water quality data is the Ministry of the Environment and Energy in cooperation with conservation authorities. Table 7.4 shows a sample of stream water quality records.

Table 7.1 (1 of 3): An AES Precipitation Record

ATMOSPHERIC ENVIRONMENT SERVICE
SERVICE DE L"ENVIRONNEMENT ATMOSPHERIQUE

RAINFALL INTENSITY-DURATION FREQUENCY VALUES
INTENSITE, DUREE ET FREQUENCE DES PLUIES

DATA INTEGRATION DIVISION
LA DIVISION DU TRAITEMENT DES DONNEES

GUMBEL - METHOD OF MOMENTS/METHODE DES MOMENTS - 1990

TABLE 1 PORT COLBORNE ONT 6136606

LATITUDE 4253 LONGITUDE 7915 ELEVATION/ALTITUDE 173 M

YEAR ANNEE	5 MIN	10 MIN	15 MIN	30 MIN	1 H	2 H	6 H	12 H	24 H
1964	8.6	13.2	14.7	28.4	34.3	45.5	56.9	56.9	64.3
1965	5.8	6.3	8.9	13.7	19.8	26.4	33.0	33.0	42.4
1966	6.9	10.7	10.7	10.7	14.0	15.0	22.1	26.2	26.7
1967	7.6	12.2	17.0	26.2	26.7	26.7	26.7	36.1	59.9
1968	8.1	14.5	16.8	19.8	26.9	42.4	81.3	101.3	112.5
1969	6.9	10.2	12.4	12.7	19.8	22.6	32.0	37.3	43.2
1970	8.4	10.9	12.2	16.0	19.3	20.3	26.4	33.3	39.6
1971	8.1	12.4	15.0	21.8	24.6	25.7	26.7	29.5	30.5
1972	5.8	9.4	13.7	17.3	23.4	23.4	27.4	33.8	36.8
1973	7.6	12.7	17.3	25.4	36.6	37.6	39.4	39.9	40.4
1974	6.9	7.9	8.6	11.7	15.2	25.7	29.7	29.7	33.0
1975	12.7	20.3	24.6	31.7	32.0	32.0	32.5	33.5	33.5
1976	4.8	7.9	9.1	11.4	19.0	23.9	23.9	38.1	47.2
1977	12.2	14.5	16.6	33.3	37.6	37.6	42.2	48.0	51.3
1978	6.9	8.8	11.1	15.5	25.7	31.6	35.5	42.0	42.0
1979	8.0	11.4	16.2	26.0	34.2	47.6	80.6	116.4	123.0
1980	11.1	14.8	15.3	17.0	25.5	32.8	33.8	41.9	44.4
1981	8.2	9.6	9.6	11.6	14.4	25.7	32.9	37.2	44.6
1982	2.2	2.2	3.2	5.1	8.4	10.6	14.4	15.6	18.0
1983	8.0	10.5	15.2	27.4	29.3	32.0	44.2	46.5	56.3
1984	9.8	15.0	18.0	26.9	28.9	30.7	30.8	51.8	54.2
1985	7.6	9.5	10.5	12.5	16.2	17.2	23.7	38.9	54.2
1986	12.4	18.4	21.2	24.7	26.5	30.6	35.1	43.0	46.6
1987	8.1	13.0	15.3	21.9	34.8	46.4	56.5	56.5	69.4
1988	8.0	11.3	12.9	14.7	17.0	20.0	22.7	42.7	47.2
1989	8.7	9.9	10.5	10.8	17.9	20.5	20.7	24.2	27.6
1990	7.2	9.0	13.1	21.8	28.0	32.8	35.9	44.9	50.4

NOTE:-99.9 INDICATES MSG DATA
DONNEES MANQUANTES

	5 MIN	10 MIN	15 MIN	30 MIN	1 H	2 H	6 H	12 H	24 H
# YRS. ANNEES	27	27	27	27	27	27	27	27	27
MEAN MOYENNE	8.0	11.4	13.7	19.1	24.3	29.0	35.8	43.6	49.6
STD. DEV. ECART-TYPE	2.3	3.6	4.3	7.4	7.8	9.5	16.2	21.1	22.9
SKEW DISSYMETRIE	0.13	0.17	0.15	0.18	-0.03	0.32	1.73	2.38	2.01
KURTOSIS KURTOSIS	4.61	4.65	4.28	2.33	2.48	2.98	6.13	9.50	7.95

WARNING / AVERTISSEMENT
YEAR 1979 HAD VALUE GREATER THAN 100 YEAR STORM.
EN 1979 L"INTENSITE DE LA PLUIE A DE PASSE
CELLE POUR UNE PERIODE DE RETOUR DE 100 ANS
DATA/LA VALEUR = 116.4 100 YEAR/ANNEE = 109.8

Table 7.1 (2 of 3): An AES Precipitation Record

ATMOSPHERIC ENVIRONMENT SERVICE
SERVICE DE L"ENVIRONNEMENT ATMOSPHERIQUE

RAINFALL INTENSITY-DURATION FREQUENCY VALUES
INTENSITE, DUREE ET FREQUENCE DES PLUIES

GUMBEL - METHOD OF MOMENTS/METHODE DES MOMENTS - 1990

TABLE 2 PORT COLBORNE ONT 6136606

LATITUDE 4253 LONGITUDE 7915 ELEVATION/ALTITUDE 173 M

RETURN PERIOD RAINFALL AMOUNTS (MM)
PERIODE DE RETOUR QUANTITIES DE PLUIE (MM)

DURATION DUREE	2 YR/ANS	5 YR/ANS	10 YR/ANS	25 YR/ANS	50 YR/ANS	100 YR/ANS	# YEARS ANNEES
5 MIN	7.7	9.6	11.0	12.6	13.9	15.1	27
10 MIN	10.8	14.0	16.1	18.8	20.8	22.8	27
15 MIN	13.0	16.8	19.4	22.6	24.9	27.3	27
30 MIN	17.9	24.5	28.8	34.3	38.4	42.4	27
1 H	23.0	29.9	34.4	40.2	44.4	48.7	27
2 H	27.5	35.9	41.4	48.5	53.7	58.9	27
6 H	33.2	47.5	57.0	69.0	77.9	86.7	27
12 H	40.2	58.8	71.1	86.7	98.3	109.8	27
24 H	45.8	66.1	79.5	96.4	108.9	121.4	27

RETURN PERIOD RAINFALL RATES (MM/HR)-95% CONFIDENCE' LIMITS
INTENSITE DE LA PLUIE PAR PERIODE DE RETOUR (MM/H)-LIMITES DE CONFIANCE DE 95%

DURATION DUREE	2 YR/ANS	5 YR/ANS	10 YR/ANS	25 YR/ANS	50 YR/ANS	100 YR/ANS
5 MIN	91.8	115.7	131.6	151.6	166.5	181.3
	+/- 9.4	+/- 15.8	+/- 21.3	+/- 28.8	+/- 34.4	+/- 40.1
10 MIN	64.5	83.8	96.6	112.8	124.7	136.7
	+/- 7.6	+/- 12.7	+/- 17.2	+/- 23.2	+/- 27.8	+/- 32.3
15 MIN	51.9	67.3	77.4	90.3	99.8	109.3
	+/- 6.0	+/- 10.1	+/- 13.7	+/- 18.4	+/- 22.1	+/- 25.7
30 MIN	35.8	48.9	57.6	68.6	76.8	84.9
	+/- 5.1	+/- 8.7	+/- 11.7	+/- 15.8	+/- 18.9	+/- 22.0
1 H	23.0	29.9	34.4	40.2	44.4	48.7
	+/- 2.7	+/- 4.5	+/- 6.1	+/- 8.3	+/- 9.9	+/- 11.5
2 H	13.7	17.9	20.7	24.2	26.8	29.4
	+/- 1.6	+/- 2.8	+/- 3.7	+/- 5.1	+/- 6.0	+/- 7.0
6 H	5.5	7.9	9.5	11.5	13.0	14.5
	+/- 0.9	+/- 1.6	+/- 2.1	+/- 2.9	+/- 3.4	+/- 4.0
12 H	3.3	4.9	5.9	7.2	8.2	9.1
	+/- 0.6	+/- 1.0	+/- 1.4	+/- 1.9	+/- 2.2	+/- 2.6
24 H	1.9	2.8	3.3	4.0	4.5	5.1
	+/- 0.3	+/- 0.6	+/- 0.8	+/- 1.0	+/- 1.2	+/- 1.4

Table 7.1 (3 of 3): An AES Precipitation Record

ATMOSPHERIC ENVIRONMENT SERVICE
SERVICE DE L"ENVIRONNEMENT ATMOSPHERIQUE

RAINFALL INTENSITY-DURATION FREQUENCY VALUES
INTENSITE, DUREE ET FREQUENCE DES PLUIES

GUMBEL - METHOD OF MOMENTS/METHODE DES MOMENTS - 1990

****

TABLE 3 PORT COLBORNE ONT 6136606

LATITUDE 4253 LONGITUDE 7915 ELEVATION/ALTITUDE 173 M

****

INTERPOLATION EQUATION / EQUATION D"INTERPOLATION: R = A * T ** B
R = RAINFALL RATE / INTENSITE DE LA PLUIE (MM /HR)
T = TIME IN HOURS / TEMPS EN HEURES

STATISTICS STATISTIQUES	2 YR ANS	5 YR ANS	10 YR ANS	25 YR ANS	50 YR ANS	100 YR ANS
MEAN OF R MOYENNE DE R	32.3	42.1	48.5	56.7	62.7	68.7
STD. DEV. R ECART-TYPE	31.4	39.8	45.4	52.4	57.7	62.9
STD. ERROR ERREUR STANDARD	7.3	9.3	10.8	12.6	14.1	15.5
COEFF. (A) COEFFICIENT (A)	19.5	26.3	30.7	36.3	40.4	44.5
EXPONENT (B) EXPOSANT (B)	-0.694	-0.669	-0.659	-0.650	-0.645	-0.641
MEAN % ERROR % D'ERREUR	9.3	8.3	8.1	7.9	7.9	7.9

Table 7.2: A Stream Flow Record

CREDIT RIVER NEAR CATARACT - STATION NO. 02HB001

DAILY DISCHARGE IN CUBIC METRES PER SECOND FOR 1990

DAY	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	DAY
1	1.12	1.34	1.41	2.15	1.47	1.29	1.25	0.957	0.934	2.03	1.12	1.57	1
2	1.07	1.52	1.41	2.37	1.42	1.15	1.12	0.854	0.894	1.64	1.14	1.48	2
3	1.05	1.21	1.34B	2.98	1.34	1.21	1.04	0.839	0.860	1.43	1.24	1.39	3
4	1.16	1.29	1.30B	2.81	1.34	1.44	0.980	0.834	0.843	1.48	1.28	2.12	4
5	1.54	1.29B	1.28B	2.68	2.10	1.32	0.939	0.889	1.01	1.77	1.58	1.89	5
6	1.36	1.24	1.26B	2.23	2.10	1.21	0.886	0.940	1.02	1.61	3.83	1.84	6
7	1.15	1.19	1.24B	2.01	1.70	1.14	0.863	0.921	1.07	1.39	2.74	1.65	7
8	1.10	1.27	1.22B	1.86	1.61	1.19	0.867	0.887	1.01	1.29	2.15	1.59	8
9	1.09	3.16	1.20	1.80	1.51	1.44	0.924	0.850	0.990	3.08	1.90	1.55	9
10	1.11	3.45	1.29	2.09	1.61	1.30	0.870	0.848	0.986	4.42	1.79	1.61	10
11	1.08	2.26	3.05	2.84	1.70	1.34	0.813	0.897	0.972	3.30	1.72	1.52	11
12	1.03B	1.83	22.4	2.49	1.55	1.19	0.854	1.14	0.941	2.73	1.66	1.49	12
13	1.04B	1.70	29.7	2.12	1.63	1.22	0.799	1.41	0.918	2.90	1.57	2.00	13
14	1.07	1.95	20.4	2.03	1.63	1.20	0.833	1.37	0.976	2.24	1.51	1.78	14
15	1.01	1.37	11.9	2.28	1.51	1.26	1.21	1.13	1.28	1.78	1.65	1.66	15
16	1.01	1.69	7.40	2.23	1.78	1.13	1.22	1.03	1.22	1.89	1.76	1.58	16
17	1.32	1.48B	5.30	2.35	2.92	1.08	1.06	0.967	1.07	2.07	1.73	1.49	17
18	5.22	1.37B	3.99	2.33	3.08	1.05	1.04	0.931	1.03	2.37	1.55	1.72	18
19	2.86	1.34	3.13	2.06	2.40	1.01	0.993	1.00	1.03	2.26	1.49	1.91	19
20	2.31	1.21	2.84	1.97	3.29	0.902	1.51	0.964	1.14	1.80	1.44	1.59	20
21	1.80	1.22	2.59	2.19	4.36	0.923	1.40	0.956	1.07	1.56	1.42	1.56	21
22	1.58	1.69	2.61	2.13	3.05	0.955	1.20	0.937	1.07	1.54	1.89	2.22	22
23	1.42	3.93	3.71	1.90	2.45	1.39	1.32	0.942	1.07	1.39	2.12	2.77	23
24	1.46	2.78B	3.03	1.72	2.16	1.73	1.21	0.916	1.06	1.31	2.01	2.10B	24
25	1.58	1.78B	2.61	1.73	1.92	1.45	1.08	0.929	1.03	1.25	1.80	1.86B	25
26	2.04	1.90B	2.39	1.81	1.74	1.20	0.978	0.876	1.03	1.22	1.66	1.99B	26
27	1.70	1.67	2.14	1.74	1.60	1.10	0.939	0.867	1.01	1.18	1.73	1.88	27
28	1.81	1.48	2.03	1.66	1.50	1.08	0.899	1.44	0.964	1.14	2.05	1.57	28
29	1.65		1.93	1.60	1.44	1.33	0.854	1.40	1.02	1.14	1.92	3.37	29
30	1.48		1.84	1.54	1.42	1.49	0.912	1.10	1.92	1.12	1.70	9.12	30
31	1.37		2.05		1.35		1.04	0.986		1.12		4.78B	31
TOTAL	47.59	49.61	149.99	63.70	60.68	36.720	31.903	31.007	31.438	57.45	53.15	66.65	TOTAL
MEAN	1.54	1.77	4.84	2.12	1.96	1.22	1.03	1.00	1.05	1.85	1.77	2.15	MEAN
MAX	5.22	3.93	29.7	2.98	4.36	1.73	1.51	1.44	1.92	4.42	3.83	9.12	MAX
MIN	1.01	1.19	1.20	1.54	1.34	0.902	0.799	0.834	0.843	1.12	1.12	1.39	MIN

SUMMARY FOR THE YEAR 1990

DISCHARGES IN CUBIC METRES PER SECOND

MEAN, 1.86
MAXIMUM DAILY, 29.7 ON MAR 13
MINIMUM DAILY, 0.799 ON JUL 13
MAXIMUM INSTANTANEOUS, 31.8 AT 06:50 EST ON MAR 13

TYPE OF GAUGE - RECORDING
LOCATION - LAT 43 50 08 N
LONG 080 01 21 W
DRAINAGE AREA, 205 km²

B - ICE CONDITIONS

REGULATED

MONTHLY TOTAL DISCHARGE IN CUBIC DECAMETRES

JAN	4 110	JUL	2 760
FEB	4 290	AUG	2 680
MAR	13 000	SEP	2 720
APR	5 500	OCT	4 960
MAY	5 240	NOV	4 590
JUN	3 170	DEC	5 760

TOTAL DISCHARGE, 58 800 dam³

Table 7.3: A Sediment Load Record

ONTARIO

AUSABLE RIVER NEAR SPRINGBANK - STATION NO. 02FF002

INSTANTANEOUS SUSPENDED SEDIMENT FOR 1990

DATE	TIME	WATER TEMP. (C)	INSTANT DISCHARGE (m³/s)	SINGLE SAMPLING VERTICAL (m)	TYPE OF SAMPLER	INSTANT CONCENT. (mg/L)	DISSOLVED SOLIDS (mg/L)	PERCENT FINER THAN INDICATED SIZE, IN MILLIMETRES .002	.004	.008	.016	.031	.062	.125	.250	.500	1.00	2.00	PERCENT SAND	SILT	CLAY
FEB 01	08:15	2.0	14.2 H	12.0	DH59	37															
FEB 23	11:20	1.5	120	12.0	DH59	302															
MAR 02	16:30	5.0	11.4	12.0	DH59	64															
MAR 15	16:00	14.0	40.5	12.0	DH59	60	229														
APR 18	10:50	8.0	13.4	12.0	DH59	67															
APR 20	09:15	9.0	10.1	12.0	DH59	73															
MAY 03	10:35	15.0	2.97	12.0	DH59	106															
MAY 31	10:20		2.78	12.0	DH59	114	283														
JUN 01	18:30	24.0	2.15	12.0	DH59	92	293														
JUN 08	15:40	23.0	2.74	12.0	DH59	89	323														
JUN 21	15:50	27.0	1.13	12.0	DH59	118	272														
JUN 28	10:00	24.0	1.81	12.0	DH59	143	328														
OCT 03	11:25		17.4	12.0	DH59	108															
OCT 05	16:30	18.0	20.0	12.0	DH59	150															
OCT 13	14:00	14.0	72.2	12.0	DH59	50	323														
OCT 23	14:00	14.0	18.8	12.0	DH59	94															
OCT 29	13:50	10.0	6.59	12.0	DH59	73															
NOV 07	16:30	10.0	93.5	12.0	DH59	91															
NOV 15	10:45	10.0	12.2	12.0	DH59	41															
NOV 22	14:50	7.0	6.77	12.0	DH59	113															
DEC 10	11:30	6.0	15.1	12.0	DH59	59															
DEC 22	13:30	10.0	21.9	12.0	DH59	48															

NOTE: SINGLE VERTICAL IS LOCATED AT 12 METRES FOR 1990

DISCHARGE SYMBOL(S): H - DAILY MEAN

AUSABLE RIVER NEAR SPRINGBANK - STATION NO. 02FF002

HISTORICAL SEDIMENT DATA SUMMARY

YEAR	PERIOD OF OPERATION SUMMARY (JFMAMJJASOND)	TYPE OF SAMPLING	SAMPLED CONCENTRATION MAXIMUM (MG/L)		MINIMUM (MG/L)		SUSPENDED SEDIMENT LOAD MAXIMUM DAILY (TONNES)		TOTAL FOR PERIOD (TONNES)	SUMMARY : JAN-DEC MEAN LOAD (TONNES)	HISTORICAL MEAN LOAD (TONNES)	STANDARD ERROR OF MEAN %
1970	CCCCCCCCCCCC	MANUAL	1 240	APR 2	10	JAN 1	4 520	MAY 16	35 200	96.4	96.4	---
1971	PPPCCCCCCCCC	MANUAL	312	AUG 28	15	DEC 23	887	APR 2	7 460	---	---	---
1972	CCCCCCCCCCCC	MANUAL	1 020	APR 17	13	FEB 27	7 790	APR 17	41 000	112	104	7.5
1973	CCCCCCCCCCCC	MANUAL	1 370	MAR 11	12	DEC 8	7 870	JAN 1	65 600	180	129	19.8
1974	CCCCCCCCCCCC	MANUAL	1 510	APR 13	10	DEC 19	15 200	MAR 5	93 600 *	256	161	22.7
1975	CCCCCCCCCCCC	MANUAL	1 710	APR 19	8	JAN 22	18 700	APR 19	75 200	206	170	17.5
1976	CCCCCCCCCCCC	MANUAL	1 240	JUL 10	11	DEC 18	5 220	MAR 6	65 500	179	172	14.2
1977	CCCCCCCCCCCC	MANUAL	848	JUN 29	23	DEC 23	6 600	MAR 13	65 500	179	173	11.9
1978	CCCCCCCCCCCC	MANUAL	653	APR 11	31	MAR 29	4 180	APR 2	39 400	108	165	11.9
1979	CCCCCCCCCCCC	MANUAL	1 860	APR 14 *	20	DEC 30	33 200	APR 14 *	92 900	255	175	11.4
1980	CCCCCCCCCCCC	MANUAL	758	JUN 29	17	FEB 14	5 890	MAR 22	41 500	113	168	11.2
1981	CCCCCCCCCCCC	MANUAL	1 040	SEP 2	10	DEC 30	4 200	SEP 2	41 300	113	163	10.9
1982	CCCCCCCCCCCC	MANUAL	1 730	MAR 31	5	JAN 31 *	18 700	MAR 31	65 700	180	165	9.9
1983	CCCCCCCCCCCC	MANUAL	712	APR 10	5	JAN 9	3 850	MAY 3	29 600	81.1	158	10.3
1984	CCCCCCCCCCCC	MANUAL	770	MAR 16	7	JAN 27	4 330	MAR 17	42 900	117	155	9.9
1985	CCCCCCCCCCCC	MANUAL	903	SEP 8	5	NOV 25	3 940	FEB 25	45 200	124	153	9.4
1986	CCCCCCCCCCCC	MANUAL	1 170	JUN 12	7	JAN 11	3 460	SEP 12	52 200	143	153	8.9
1987	CCCCCCCCCCCC	MANUAL	379	MAR 9	12	MAR 29	4 130	MAR 9	37 000	101	150	8.7
1988	--CCCCCCCCCC	MANUAL	465	MAR 25	13	MAR 1	2 550	MAR 25	25 400	---	---	---
1989	CCCCCCCCCCCC	MANUAL	319	MAR 18	11	MAR 4	1 180	NOV 16	15 900	43.7	144	9.5
1990	PCCCCCCCCCCC	MANUAL	553	JUL 11	8	FEB 21	3 250	FEB 23	60 200	---	---	---

MANUAL - MANUAL SAMPLING

C - COMPLETE MONTH P - PARTIAL MONTH * - EXTREME FOR PERIOD OF RECORD

AUX SABLES RIVER AT MASSEY - STATION NO. 02CE002

LOCATION: LAT 46 12 54 N LONG 082 04 14 W
DRAINAGE AREA: 1 350 km²
TYPE OF FLOW: REGULATED
TYPE OF GAUGE: RECORDING

INSTANTANEOUS SUSPENDED SEDIMENT FOR 1990

DATE	TIME	WATER TEMP. (C)	INSTANT DISCHARGE (m³/s)	SINGLE SAMPLING VERTICAL (m)	TYPE OF SAMPLER	INSTANT CONCENT. (mg/L)	DISSOLVED SOLIDS (mg/L)	PERCENT FINER THAN INDICATED SIZE, IN MILLIMETRES .002	.004	.008	.016	.031	.062	.125	.250	.500	1.00	2.00	PERCENT SAND	SILT	CLAY
MAY 08	10:39	10.0	26.5	12.0	DH59	30															

NOTE: SINGLE VERTICAL IS LOCATED AT 12.0 METRES FOR 1990

Table 7.4 (1 of 2): A Stream Water Quality Record

1990 WATER QUALITY DATA REGION 6

B.O.M./ SITE: NABIGOON RIVER — STATION ID: 19-0001-036-02
SAMPLE POINT: HIGHWAY 609 NEAR QUIBELL
STATION TYPE: RIVER — FLOW GAUGE FED 05QD006

MAJOR BASIN: ARCTIC DRAINAGE NELSON RIVER — STORET CODE: 05
MINOR BASIN: LAKE WINNIPEG EAST WINNIPEG RIVER — 001
TERM STREAM: ENGLISH RIVER — 1890

LAT: 49 57 29.81 LONG: 093 24 01.63 U T M: 15 0471275.0 5533850.0 4 REGION: 06 DISTANCE: 199.875

M=INTERIM TEST-NAME:		FGPROJ	ALKT	BOD5	CLIDUR	COLTR	COND25	CUUT	DO	ECMPN	FEUT
SAMPLE DATE YYMMDD	HOUR LMT	PROJECT SUB-PROJ CODE	ALK TOTAL MG/L AS CACO3	BOD 5 DAY TOT.DEM. MG/L AS O	CHLORIDE UNF.REAC MG/L AS CL-	COLOUR TRUE TCU	CONDUCT. 25C UMHO/CM AT 25 C	COPPER UNF.TOT. UG/L AS CU	DISOLVED OXYGEN MG/L AS O	E.COLI BY MPN CNT /100ML	IRON UNF.TOT. UG/L AS FE
SAMPLE NUMBER											
900108 1530	21003	0101	69.0	2.5	61.0	211.0	379.00	5.0<W	12.20	43	600
900220 1515	21008	0101	70.0	3.1	73.0	275.0	413.00	1.7<T	14.00	93	460
900305 1440	21013	0101	67.0	1.6	61.0	178.0	355.00	5.1<T	13.40	43	450
900514 1515	21018	0101	63.0	1.4	42.0	177.0	285.00	7.8	10.60	20	550
900611 1520	21023	0101	52.0	1.3	9.7	95.0	146.00	1.6<T	8.00	23	800
900716 1700	21028	0101	52.0	2.5	16.0	85.0	162.00	0.9<T	8.30	15	390
900813 1530	21033	0101	48.0	1.3	23.0	36.0	180.00	1.4<T	7.90	23	350
900910 1650	21038	0101	50.0	1.2	2.0	38.0	109.00	2.4<T	8.00	23	590
901015 1610	21043	0101	59.0	2.2	37.0	53.0	252.00	1.5<T	10.30	9	620
901113 1620	21047	0101	50.0	2.3	37.0	57.0	243.00	2.1<T	14.00	7	400
	MAXIMUM		70.0	3.1	73.0	275.0	413.00	7.8	14.00	93	800
	ARITH MEAN		58.0	1.9	36.2	120.5	252.40	2.9<A	10.67	30	521
	GEOM MEAN		57.4	1.8	25.4	95.2	231.91	2.4<A	10.39	23	506
	MINIMUM		48.0	1.2	2.0	36.0	109.00	0.9	7.90	7	350
	STD DEV (GEOM M)		8.6	0.7	23.7	83.7	104.77	2.2<A	2.57	2M	137
	SAMP IN STATISTICS		10	10	10	10	10	10	10	10	10
	% SAMP (EXCLUDED)										

M=INTERIM TEST-NAME:		FMSTRC	FWTEMP	HARDT	HGUT	MNUT	NIUT	NNHTFR	NNOTFR	NNO2FR	NNTKUR
SAMPLE DATE YYMMDD	HOUR LMT	STREAM COND.	WATER TEMP DEG.C	HARDNESS TOTAL MG/L AS CACO3	MERCURY UNF.TOT. UG/L AS HG	MANGANSE UNF.TOT. UG/L AS MN	NICKEL UNF.TOT. UG/L AS NI	NH3-N TOTAL FIL.REAC MG/L AS N	NO2+NO3N FIL.REAC MG/L AS N	NO2-N FIL.REAC MG/L AS N	K'DAHL N TOTAL UNF.REAC MG/L AS N
SAMPLE NUMBER											
900108 1530	21003	4	0.5	85	0.013<T	95.0	10.0<W	0.26	0.070	0.016	1.200
900220 1515	21008	4	0.5	85	0.015<T	100.0	1.0<W	0.25	0.090	0.019	1.140
900305 1440	21013	4	0.5	84	0.012<T	89.0	10.0<W	0.29<=>	0.125<=>	0.015	1.080
900514 1515	21018		11.0	80	0.018<T	46.0	1.5<T	0.20	0.125	0.021	0.950
900611 1520	21023	3	17.0	57	0.066	35.0	1.0<W	0.02<T	0.030	0.009	0.760
900716 1700	21028	6	22.5	58	0.022<T	26.0	1.0<W	0.01<W	0.045	0.009	0.650
900813 1530	21033	6	17.0	53	0.018<T	31.0	1.0<T	0.04	0.110	0.012	0.580
900910 1650	21038	6	17.0	52	0.029	13.0	1.0<W	0.03<T	0.005<T	0.003<T	0.500
901015 1610	21043		7.8	67	0.036	36.0	1.0<W	0.10	0.040	0.010	0.700
901113 1620	21047	6	0.5	59	0.024<T	44.0	1.0<W	0.08	0.040	0.010	0.690

Table 7.4 (2 of 2): A Stream Water Quality Record

1990 WATER QUALITY DATA REGION 6

B.O.W./ SITE: WABIGOON RIVER
SAMPLE POINT: HIGHWAY 609 NEAR QUIBELL
STATION TYPE: RIVER FLOW GAUGE FED 05QD006

STATION ID: 19-0001-036-02

MAJOR BASIN: ARCTIC DRAINAGE NELSON RIVER
MINOR BASIN: LAKE WINNIPEG EAST WINNIPEG RIVER
TERM STREAM: ENGLISH RIVER

STORET CODE: 05 001 1890

LAT: 49 57 29.81 LONG: 093 24 01.63 U T M: 15 0471275.0 5533850.0 4 REGION: 06 DISTANCE: 199.875

W=INTERIM TEST-NAME:	FWSTRC	FWTEMP	HARDT	HGUT	MNUT	NIUT	NNHTFR	NNOTFR	NNO2FR	NNTKUR
SAMPLE DATE HOUR YYMMDD LMT / SAMPLE NUMBER	STREAM COND.	WATER TEMP DEG.C	HARDNESS TOTAL MG/L AS CACO3	MERCURY UNF.TOT. UG/L AS HG	MANGANSE UNF.TOT. UG/L AS MN	NICKEL UNF.TOT. UG/L AS NI	NH3-N TOTAL FIL.REAC MG/L AS N	NO2+NO3N FIL.REAC MG/L AS N	NO2-N FIL.REAC MG/L AS N	K'DAHL N TOTAL UNF.REAC MG/L AS N
MAXIMUM		22.5	85	0.066	100.0	10.0	0.29	0.125	0.021	1.200
ARITH MEAN		9.4	68	0.025<A	51.5	2.8<A	0.13<A	0.068<A	0.012<A	0.825
GEOM MEAN		3.8	67	0.022<A	43.2	1.7<A	0.08<A	0.051<A	0.011<A	0.792
MINIMUM		0.5	52	0.012	13.0	1.0	0.01	0.005	0.003	0.500
STD DEV (GEOM M)		8.6	14	0.016<A	31.3	3.8<A	0.11<A	0.043<A	0.005<A	0.248
# SAMP IN STATISTICS		10	10	10	10	10	10	10	10	10
% SAMP (EXCLUDED)										

W=INTERIM TEST-NAME:		PH	PPO4FR	PPUT	RSP	RST	SSO4UR	TCMF	TURB	ZNUT
SAMPLE DATE HOUR YYMMDD LMT	SAMPLE NUMBER	PH	PO4 FIL.REAC MG/L AS P	PHOSPHOR UNF.TOT. MG/L AS P	RESIDUE PARTIC. MG/L	RESIDUE TOTAL MG/L	SULPHATE UNF.REAC MG/L AS SO4	COLIFORM TOTAL MF CNT /100ML	TURB'ITY FTU	ZINC UNF.TOT. UG/L AS ZN
900108 1530	21003	7.40	0.015	0.083	10	270	20.40	19000	13.00	22.0<T
900220 1515	21008	7.50	NO DATA|BC	0.083	10	290	13.70	NO DATA|OP	14.00	9.7
900305 1440	21013	7.40	0.025<=>	0.079	8	260	15.10	19000J	13.00	15.0<T
900514 1515	21018	7.60	0.034	0.074	9	220	9.32	600A	15.00	9.1
900611 1520	21023	7.50	0.011	0.054	15	140	3.88	26000	22.00	2.0<T
900716 1700	21028	7.50	0.011	0.046	8	140	5.86	100A	12.00	1.4<T
900813 1530	21033	7.60	0.019	0.053	6	130	6.34	100<	8.70	1.9<T
900910 1650	21038	8.20	0.007	0.026	7	100	2.86	100<	8.90	2.1<T
901015 1610	21043	7.40	0.004<T	0.047	20	190	8.40	100<	7.00	3.8<T
901113 1620	21047	7.50	0.011	0.045	10	190	11.40	200A	11.10	4.6<T
MAXIMUM		8.20	0.034	0.083	20	290	20.40	190000	22.00	22.0
ARITH MEAN		7.56	0.015<A	0.059	10	193	9.73	39317	12.47	7.2<A
GEOM MEAN		7.56	0.013<A	0.056	10	183	8.30		11.89	4.7<A
MINIMUM		7.40	0.004	0.026	6	100	2.86	100	7.00	1.4
STD DEV (GEOM M)		0.24	0.009<A	0.020	4	66	5.49		4.20	6.9<A
# SAMP IN STATISTICS		10	9	10	10	10	10	6	10	10
% SAMP (EXCLUDED)								33		

Field Investigation

Introduction

Field investigation is an important component of the data collection process in highway drainage management, for example, in the hydraulic design of bridges and culverts. The *Ontario Highway Bridge Design Code* recognizes this fact and requires that field investigation must be carried out for the design of bridges and large culverts.

A field investigation is useful to:

- Familiarize designers with the site conditions.
- Collect raw and supplemental data.
- Verify existing data.
- Identify and investigate areas of concern.
- Plan for a detail survey or special purpose survey.

Members of a field investigation team may be involved in indoor or outdoor activities or a combination. A team is often multidisciplinary just as a design team is. Members may have background in highway design, geotechnical engineering, structural engineering, environmental planning, biology, surveying, highway maintenance and operation, health and safety, in additional to hydrology and hydraulics.

Field Investigation Process

A field investigation requires a substantial amount of time, money, labour and equipment. There is a substantial amount of data in the field, it is important to ensure that a sufficient amount of data is collected and an appropriate level of effort is used.

- The type and amount of data to be collected should be determined as much as possible prior to a field trip. The data requirements will affect the type of equipment needed, length of time required, number of persons involved and details of trip arrangements.
- Appropriate levels of effort should be spent to appropriate amounts of data. Constraints, such as insufficient time, not enough people or out-of-date equipment can curtail the necessary scope of field work. On the other hand, an excess scope of field work may strain the available resources.

It is essential to plan and carry out a field investigation in a systematic way to avoid unsatisfactory results. There are many reasons why results can be unsatisfactory such as:

- Some necessary equipment such as a measuring tape, plans, maps, or camera, is not at hand.
- Access routes have not been adequately checked. Roads may be inaccessible.
- Poorly equipped with safety gear or protective clothing.
- Itinerary has not been thoroughly planned - insufficient time, etc.
- Not aware of the extent and details of data requirements and therefore, insufficient data is collected.
- Equipment failure.
- Unexpected or unprepared adverse weather conditions.
- Timing of the operation - certain operations such as investigation of ice problems can only be carried out in a particular season and have to be planned ahead.

A field investigation thoroughly planned and carried out will greatly expedite the investigation and ensure that no important data is missed. Figure 7.4 shows a field investigation process that may be followed. Users should note that there is no single standard process that can be used to cover all situations. In special or unusual circumstances, users should take a cautious approach and be flexible in the planning of a field trip. A brief discussion of the key steps of this process follows.

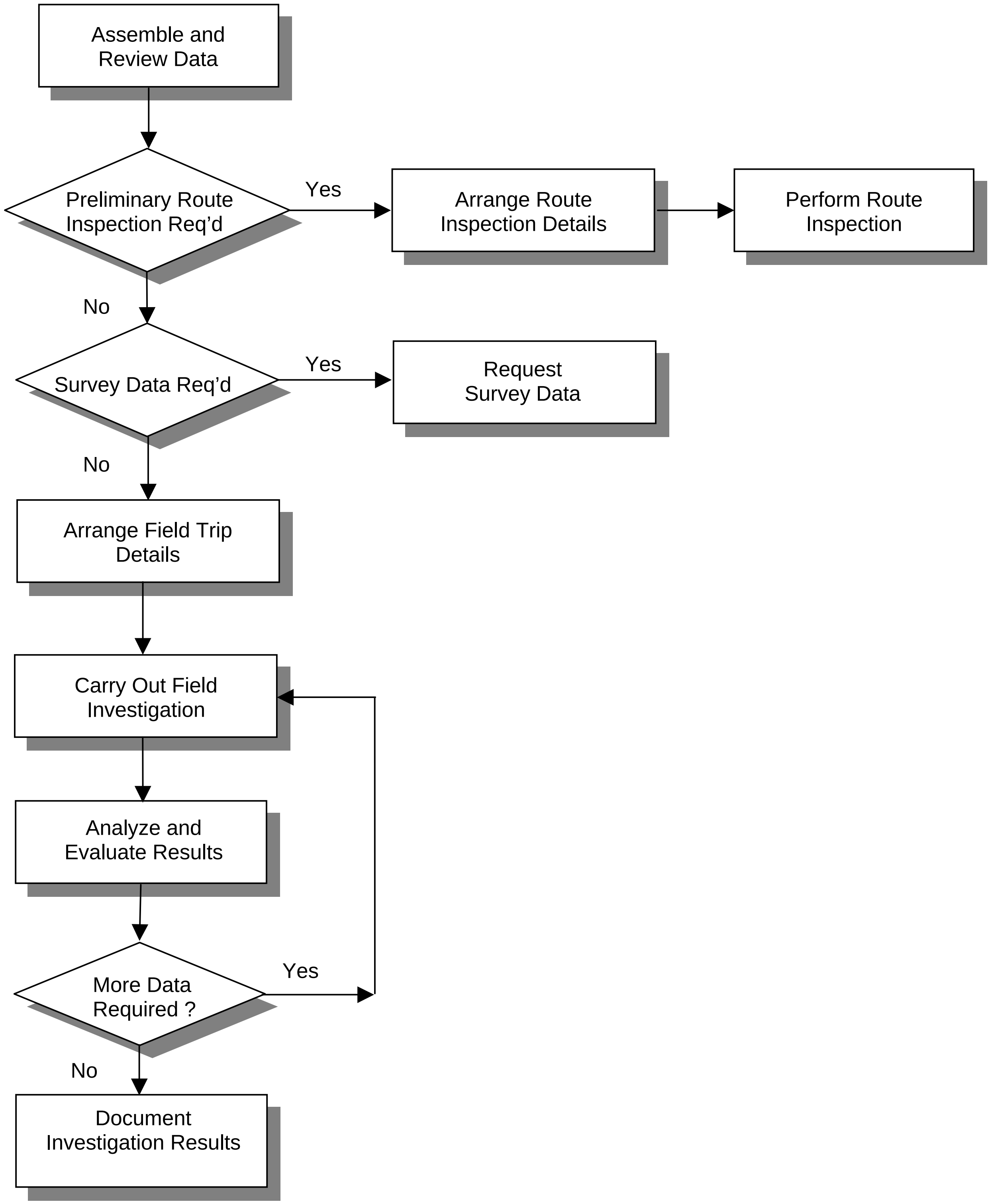
Assemble and Review Data

As a first step, a study is necessary to determine the types and amount of data to be collected. It involves the assembly and review of maps, plans, airphotos, charts, records, reports, etc. This step determines the extent of data required to be collected and also existing data to be verified in the field. This step, however, does not define the data requirements of a project. Data requirements are dealt with in the chapters of this manual where design subjects are discussed.

Usually a lot of data is available from various sources (See the section Primary Data Sources). The data to be assembled and reviewed typically include:

- Watershed plans, flood plain maps and site plans.
- Stormwater management reports, etc.
- Topographic maps.
- Land use maps.
- Geology and soil maps.
- Airphotos and mosaics.
- Climatic records.
- Stream flow records.
- Highway engineering drawings.
- Fish habitat and wildlife information.
- Current and past reports.

Figure 7.4: Flow Chart of Field Investigation Process



Preliminary Route Inspection

The previous step and the degree of the investigators' familiarity with the site would give an indication whether a route inspection is necessary prior to the actual field investigation. The purpose of a preliminary route inspection is to verify the practicality and completeness of the details of the field investigation plan.

Possible activities during a preliminary route inspection:

- Traverse the entire route or proposed location of work.
- Note the topography and site terrain.
- Record drainage routes and potential drainage related problems, such as erosion and scour.
- Note natural features and constructed structures.
- Take photographs of selected features and site surroundings.
- Record potential obstructions and potential problem areas.
- Note existing roads, access, etc. and record any problems.
- Establish initial contacts with local residents and make appointments for getting their assistance or making interviews with them.

Request Site Survey

This may be a good time in the field investigation process to request a site survey if one is needed. A possible alternative timing would be concurrently with the field investigation or soon after it. It will be useful to coordinate the objects of the field investigation and those of the survey.

Arrange Field Trip Details

Determine the objects that will be observed or measured during the field trip, using the section Objects of Field Investigation later in this chapter as a guide. There are also many other details to be worked out. Appendix 7B provides some suggestions for preparing for these details. The arrangements should be agreed to/understood by team members before they set out for the trip. This point is particularly important in health and safety matters.

Carry Out the Field Investigation

Carry out the field trip according to the prepared plan. Record observations and measurements and take photographs along the journey and discuss any points of interest or questions with teammates to ensure that important information is captured properly. If a change of course of action is required because of an unexpected circumstance, make the necessary change according to the agreed field investigation procedure. Adhere to formal health and safety procedure of the office.

Analyze and Evaluate Results

When memory is still fresh after a field trip, carefully review field notes and photographs. This step should be done in parallel with the information obtained from the Assemble and Review Data step. Information gaps and uncertainties should be identified and clarified among team members if possible. A further trip(s) may be necessary to remove such gaps and uncertainties if the missing data could not be obtained by other sources. Sometimes, unresolved matters and uncertainties may be deferred until another field investigation, if one is expected.

In archiving the photographs, carefully identify the subjects and their locations. Record the dates and camera directions of the photographs together with appropriate annotations. The negatives or slides should be filed with field notes for future reference.

Document Investigation Results

Finally document the results in such a way that they will remain decipherable and comprehensible to future readers who may not have taken part in the field investigation. A report may be prepared to sum up available information, point out missing data, request survey data, describe potential problems, etc.

Objects of Field Investigation

Besides general familiarization with site conditions, a field investigation is usually carried out to supplement missing information or to verify uncertainties on some of the following items required in a hydrologic or hydraulic analysis or design.

- Channel roughness.
- Water levels.
- Existing structures on flood plain.
- Existing water crossings.
- Flood path and relief flow.
- Aggradation, degradation and artificial deepening.
- Habitat data.
- Scour at pier and culvert foundations.
- Ice jams.
- Debris.
- Dams and other stream controls.
- Beaver dams.
- Stream geomorphologic features.

Channel Roughness

In an important hydrologic and hydraulic analysis, field investigation may be carried out to clarify the type of channel roughness to be used if there are uncertainties in the available desktop data.

Observations/measurements to be made:

- The types and sizes of lining material in the channel bed and banks, for example, silt, sand, gravel or rock, etc. If the lining is vegetation, the type of vegetation; for example, undergrowth, weeds, trees, long grass, row crops, bushes, etc.
- Direction of growth, for example, row crops with the rows parallel to the direction of flow.
  - Thickness of growth - light or dense.
  - Height of vegetation.
  - Condition - cut or uncut.
  - Whether shrubs and small trees are likely to be bent over and submerged during floods, thereby influencing their effect on retarding flow.
  - Whether there are significant changes in the vegetation characteristics.

Consideration should be given to the seasonal presence of grown vegetation although a particular crop may not be present during a site visit. The worst scenario case of fully grown vegetation should be considered as it will decrease flow conveyance and increase flow depth.

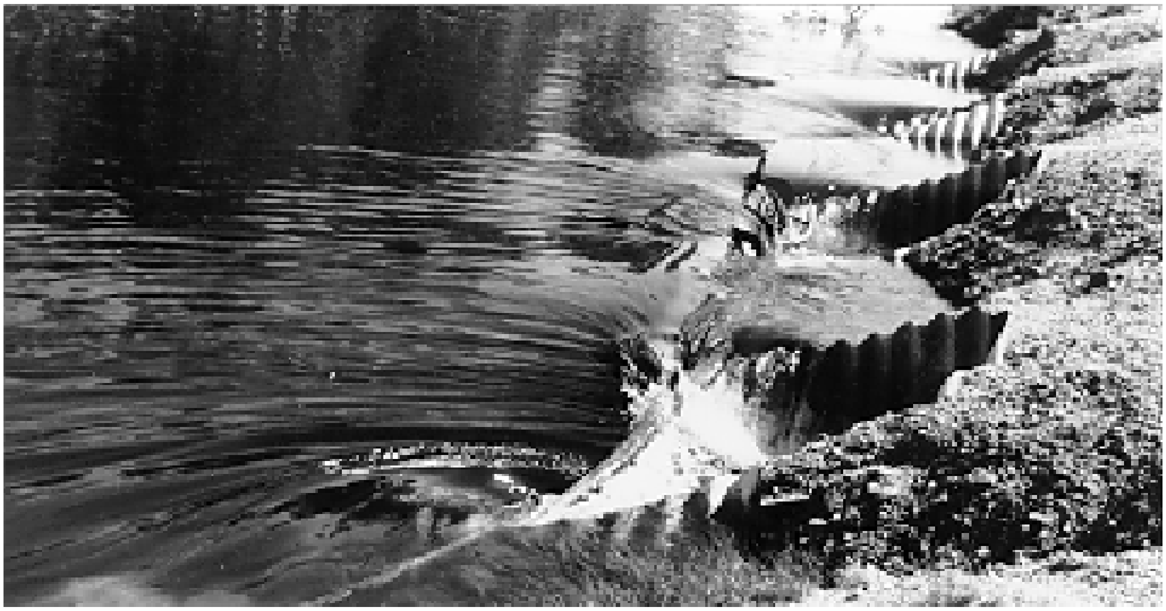
Water Levels

Observed water levels may be used to verify calculated levels or calibrate a calculation method. Wherever possible, obtain the elevations of two highest floods. The reason for noting the second highest flood is that it provides some indication of whether the highest one was due to an extreme flood. If the worst flood is much higher than the second worst, further investigations should be made to ascertain whether it was indeed the worst or an exceptional flood such as that caused by a hurricane. Two flood elevations at two separate locations should be noted, if possible, to provide a cross reference check.

The most reliable flood elevation marks are those recorded on a permanent object such as a bridge abutment wall. High water marks on both upstream and downstream sides of a structure should be noted. The difference between the two marks provides an indication of the extent of backwater effect. See Figure 7.5.

Because many flood marks soon become untraceable, they should be recorded as soon as possible after the flood event. If levels cannot be measured immediately, floodlines may be marked by stakes or paints, etc. Floodlines can also be identified on airphotos taken at the time of the flood.

Figure 7.5: Flood Levels at a Culvert Crossing



Existing Structures on Flood Plain

Existing structures on a flood plain may be affected by the proposed drainage management for the highway project or the existence of the structures may influence the drainage design.

Observations/measurement to be made:

- The structures upstream and downstream of the highway potentially requiring considerations in a hydrologic or hydraulic analysis. Their positions relative to flood paths.
- Elevations of critical points on the structures (main floors, basement window sills, etc.).
- Types of usage (industrial, sewage treatment plant, cottage, etc.). Age and value of the structures ($100,000 or $1 million).
- Any flood marks, ice scars, scour marks, etc. on the walls.
- Surrounding features, slope of a bank, erodibility of the soil, etc.
- Occupants of a building may have local knowledge regarding the history of the building. Note their names and phone numbers for possible future contact.

Existing Water Crossings

Information on existing water crossings may be a useful guide for designing a new crossing on the same watercourse.

Observations/measurements to be made:

- Most of the items as discussed under Existing Structures on Flood Plain.
- Response of the stream at the water crossing (smooth or sharp turn of flow direction, change in flow velocity, adjacent bank conditions, etc.).
- Historical signs of performance problems of the crossing (unintended relief flow, ice jam, insufficient freeboard, etc.). See also applicable discussions under the other objects of field investigations, for example, Aggradation and Degradation; Relief Flow, etc.
- Figure 7.6 shows an example of site measurements of a water crossing.

Flood Path and Relief Flow

High flows in a stream may overflow onto an adjacent flood plain, especially in flat terrain. This relief flow may be a design provision or an unintended occurrence. A water crossing downstream of the overflow point will not experience the full rate of the high flow. This fact, if known to the hydraulic engineer, may influence the choice of design flow values for hydraulic analysis of the water crossing. Field observations may verify whether overflow did or may occur.

Observations/measurements to be made:

- Signs of flood path on a flood plain (See Water Levels above).
- Signs of relief flow at the site (an opening in the bank of the watercourse, eroded path or ditch originating from a point of the bank, flood marks on trees beyond the bank, etc. See also Water Levels above).
- Details of existing relief flow structures and any signs of scour or sedimentation around the inlets and outlets of the structures.
- Trace the flood paths and relief flow paths with the aid of airphotos, if available, which show such paths. See Figures 7.7, 7.8, 7.9 and 7.10.

Figure 7.6: A Sample of Bridge Opening Measurements

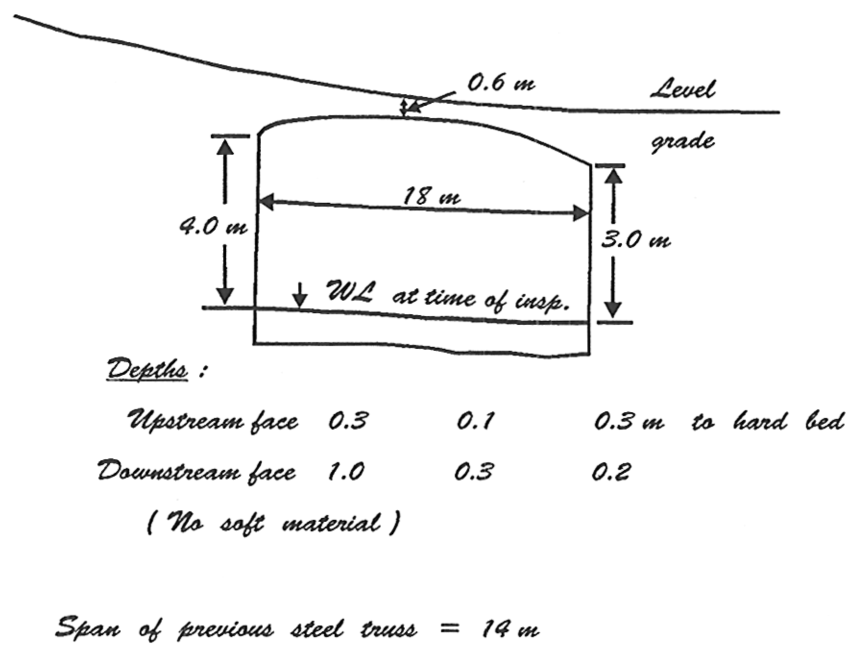


Figure 7.7: Cross Country Flow Bypassing a Bridge Site

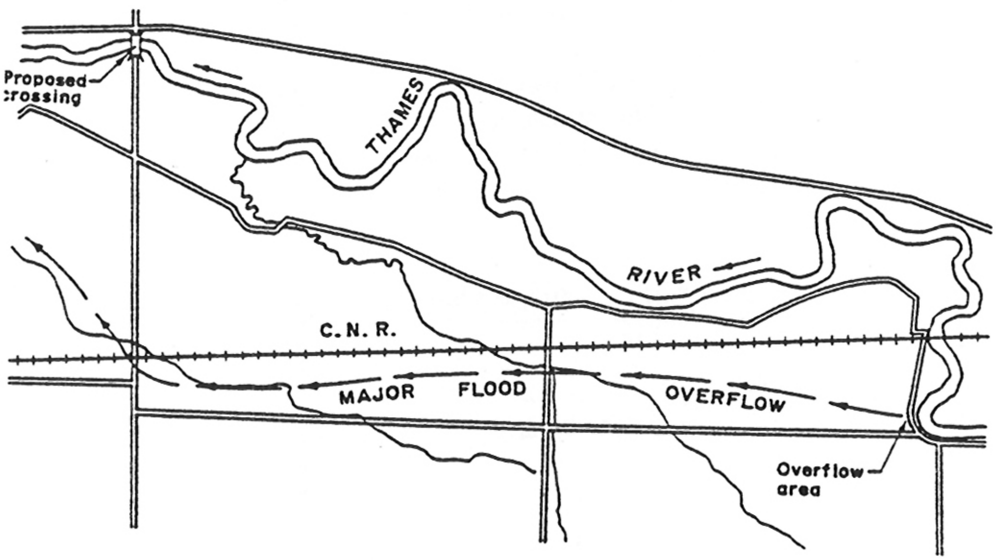


Figure 7.8: Flow Pattern Seen in Airphoto

Figure 7.9: Stream Overflow Seen in Airphoto

Figure 7.10: Evidence of Relief Flow at a Culvert

Aggradation, Degradation and Artificial Deepening

Airphotos are very useful in assessing stream channel instability. Field investigation can be carried out in parallel with airphotos in identifying unstable banks, degradation and aggradation. Unstable streams may be recognized by eroding banks, progressive degradation or aggradation, wide unvegetated point bars or recent meander cutoffs.

Observations/measurements to be made:

- Evidence of degradation, scour and erosion at stream crossings. See Figure 7.11.
- Whether there are any check dams on the stream that may have been constructed to overcome a degradation problem. If there is some evidence that the general bed level has lowered since the check dams were built, degradation is a likely cause. See Figure 7.12.
- Whether there are any other structures near the site, such as retaining walls and whether the general bed level has lowered since they were built. See Figure 7.13.
- Whether there is any evidence of head cutting of the stream bed, or whether the channel is eroding both at its bed and banks, particularly on small streams which have been extensively straightened or which ultimately flow over the edge of a valley or bluff.
- Evidence of aggradation such as extensive new gravel deposits or an unstable channel.
- Evidence of straightening, diking, dumping of excavated material or dredging in the stream.
- Whether there are any drop structure, chutes, flumes, spillways or check dams at or near a crossing which indicate past degradation problem. See Figure 7.14.
- Whether the general level of the stream bed appears to have lowered since the bridge or

culvert was built. See Figure 7.15. Care should be taken to distinguish between degradation and artificial deepening.

- Whether a concrete or other floor has been added to the structure or whether there is a significant drop in the stream bed. See Figure 7.16.
- The type of material of the channel bed and banks.
- If the bed consists of cobbles or boulders, check whether it is a veneer overlying more scourable soil. If the bed is soft, measure the thickness of soft material by probing with a range pole.
- Visual observations of the size and gradation of underlying noncohesive soils and consistency of clays, tills and other cohesive soils. Collect soil samples for analysis if necessary.
- If bedrock is suspected, note whether a soil investigation is needed.

Figure 7.11: Stream Bank Erosion

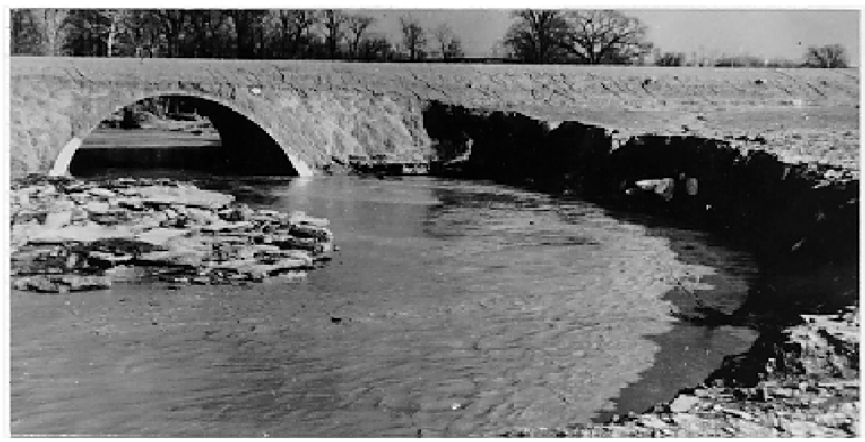


Figure 7.12: Undermined Check Dam Indicating Degradation

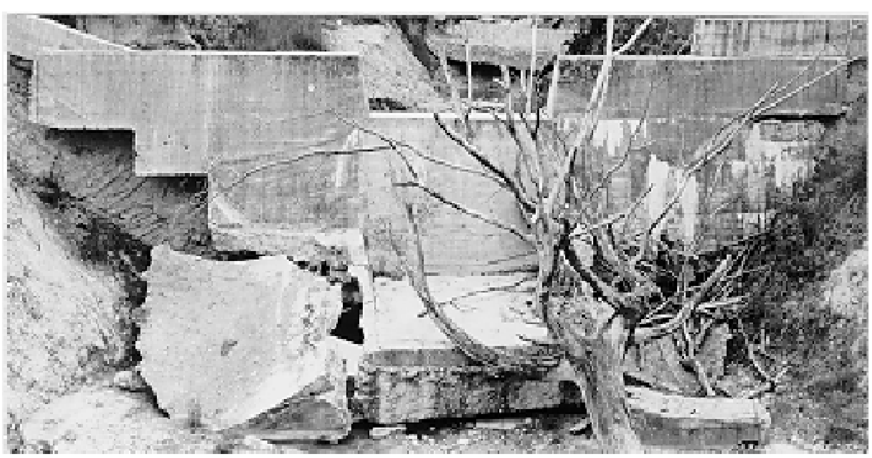


Figure 7.13: Stream Bed Degradation Exposing Retaining Wall Footing

Figure 7.14: Undermined Flume Originally Constructed to Control Degradation at Culvert Upstream

Figure 7.15: Degradation at Bridge Pier Footing

Figure 7.16: Degradation at Recently Added Concrete Floor and Cutoff

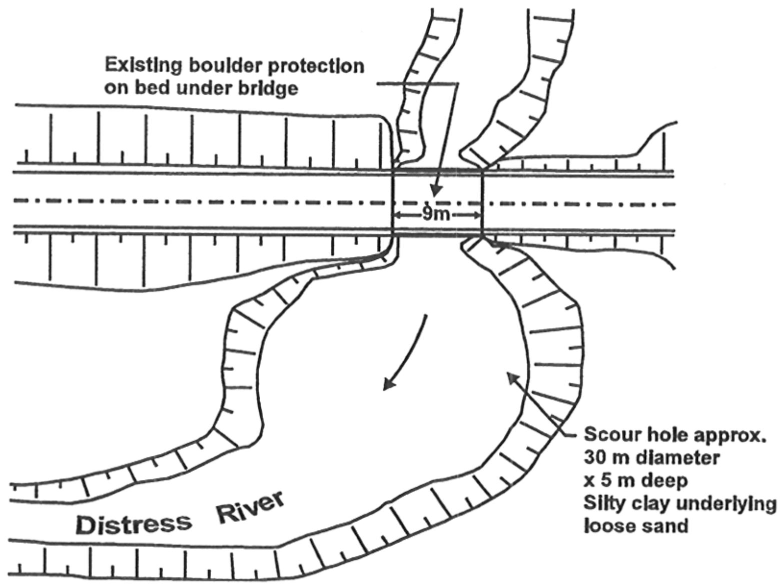
Habitat Data

As habitat data is of a specialized nature, field investigation should be done together with a fishery/wildlife biologists. After the data requirements are determined and available data reviewed by the team, use the discussions under the various objects of field investigation as a guide to collect the hydrologic and hydraulic data required for compiling the complete habitat data.

Scour at Pier and Culvert Foundations

At a bridge or culvert, check for obvious signs of scour such as a deep scour hole. See Figure 7.17. Damage or repairs to foundations of bridge piers or culverts are possible evidence of scour. Sounding or other underwater measurement techniques is usually required to detect scour.

Figure 7.17: Scour Hole Produce by High Velocity Flow through Culvert



Observations/measurements to be made:

- Significant cracking of piers or abutments. See Figure 7.18.
- Erected steel sheet piling, particularly if it is only partly driven into the substructure.
- Undermined footings, headwalls, culvert ends etc.
- Underpinning of the substructure.
- Tilting or settlement of piers or abutment. Settlement of abutment may have been repaired by building up the abutment. See Figure 7.19.
- Piers or abutments not of the same age.
- Riprap dumped under a bridge or culvert, often forming a rapid flow section.

- Concrete floor laid under a bridge or culvert. See Figure 7.20.
- Scour protection works at relief structures.
- If scours holes are detected, collect preliminary data for determining whether further Investigation is required.

Figure 7.18: Crack in Culvert due to Scour

Figure 7.19: Damage in Bridge Abutment due to Scour

Figure 7.20: Scour Counter-measure Using Concrete Flooring

Ice Jams

The time spent on collecting ice data should be related to the importance of the crossing and the severity of ice problems likely to occur. Small culvert crossings, for example an 800 mm diameter pipe, usually require little or no investigation.

The occurrence of past ice jams is virtually impossible to detect by inspection. Therefore, it is important to rely on local information obtained from land owners and local offices of government agencies.

Observations/measurements to be made:

- The height, size and depth of ice scars on trees. See Figure 7.21.
- Evidence of soil pushed up the banks by ice shove or ice gouges in the banks or on the flood plain.
- Probable locations of jams, for example, at bends, constrictions, etc.
- The average and maximum dimensions of ice floes and their average and maximum thickness; their composition, for example, green ice, compressed snow, slush, etc. See Figure 7.22.
- The velocities of moving ice floes.
- The location and direction of an overflow on the flood plain caused by ice jams.
- The height and extent of backwater caused by the jam.
- The possible cause of ice jam, for example, unbroken ice sheet, ice sheet frozen to stream bed, etc. See Figure 7.23.

Figure 7.21: Ice Scars on Tree

Figure 7.22: Loosely Packed Ice Floes

Figure 7.23: Large Ice Jam

Debris

Occasionally, debris can gather at a small bridge or the inlet of a culvert. See Figure 7.24. The type, size and amount of debris should be noted. The debris carrying potential of a stream should be estimated. In some cases, debris marks can be observed on tree.

Dams and Other Stream Controls

A downstream dam may affect the water levels and increase the backwater at a crossing. A dam immediately upstream may cause serious erosion problems or even a failure if the dam were to wash out. See Figure 7.25.

Observations/measurements to be made:

- The location of a dam at or next to a crossing, and the direction of its spillway.
- The type of construction and condition of the dam and its approach channels.
- The following information if the dam discharge is to be calculated:
    - The details as shown in Figure 7.26; the shape of the weir crest, abutments and piers.
    - Details of stop logs, gate openings, debris, etc.
    - Information on any known flow through turbines or bypass channels that would be added to the flow over the dam.
    - Details of past washouts of the dam or its approaches.

Figure 7.24: Debris Jam at a Multi-span Culvert

Beaver Dams

Observations/measurements to be made:

- The location of beaver dams and their approximate heights and sizes and water levels. Figure 7.27 shows a beaver dam at the inlet of a culvert. Figure 7.28 shows a beaver dam washout.
- The ownership of the land on which the beaver dams are built.
- Any flood or other damage likely caused by flows from the dams or dam breaks.
- The duration of existence of the dams and any history of dams being rebuilt by beavers after destruction by human control efforts.

Stream Geomorphologic Features

A field investigation on stream geomorphologic features can be complex and wide-ranging and is beyond the scope of this chapter. Refer to Robert Newbury, *Stream Analysis and Fish Habitat Design, a Field Manual* for guidance.

Figure 7.25: Earth Fill Dam Overflowing

Figure 7.26: Information for Calculating Discharge over a Dam Spillway

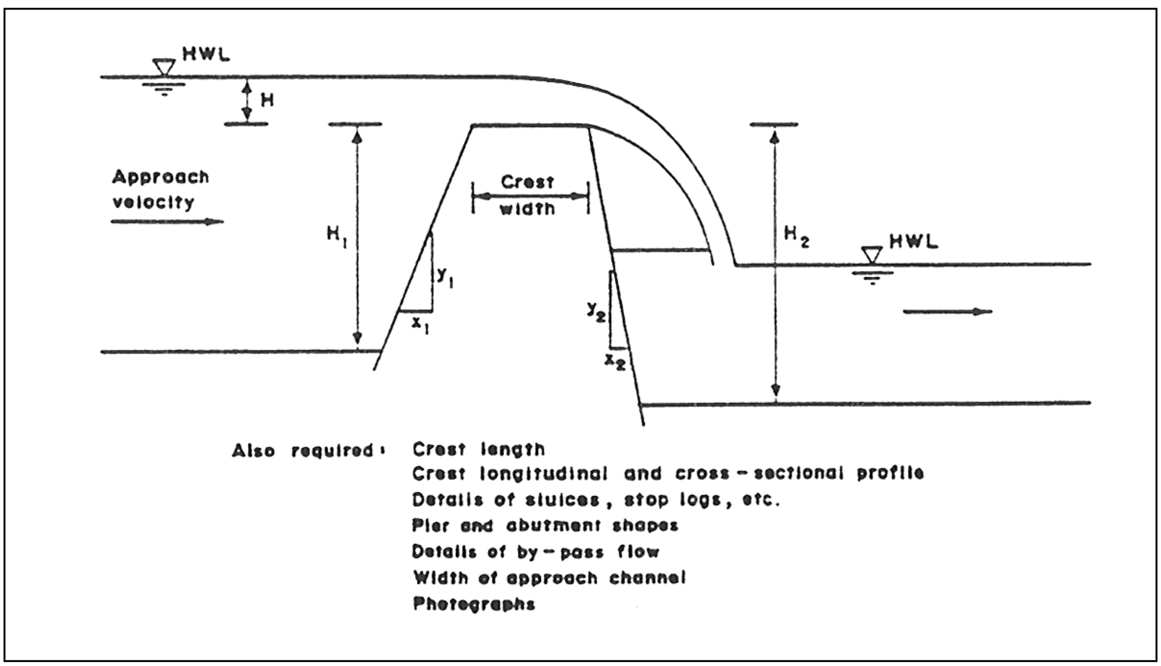


Figure 7.27: Beaver Dam at Inlet of Culvert

Figure 7.28: Beaver Dam Washout

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Appendix 7A: Tables of Data Sources

Table 7A.1 (1 of 3): Primary Data Sources - General Listing

Data Type	Data Sources and Description
Topographic Data	• MNR - Ontario Base Maps. • Natural Resources Canada (NRC) - National Topographic Systems (NTS). • Examples of topographic information: Ground terrain. Existing structures. Drainage patterns. Water bodies. Spot elevations and contours.
Land Use and Ground Cover	• Municipalities - Official Plans of existing and future land uses. • OMAFRA - Agricultural land uses. • Conservation authorities - Watershed management plans including land use information. • Conservation authorities - floodline and fill mapping. • MOEE - Sewage treatment plants: name, location, owner, operator, receiving water, effluent characteristics. • MOEE - Water supply plants: name, location, owner, operator, supply sources and quality. • Topo maps and aerial photographs also contain existing land use information.
Airphotos	• MNR - Airphotos of various scales and dates. May provide data similar to maps through photogrammetric and remote sensing applications.
Soil and Groundwater Well Data	• Agricultural Canada in cooperation with OMAFRA - "County" soil maps. • OMAFRA - Agricultural soils data. • MNR - Surficial geology maps of northern Ontario. • Canada Department of Energy, Mines and Resources - Land classification maps. • MOEE - A database of groundwater wells.
Precipitation and Other Climatic Data	• Environment Canada - A set of maps on Canada's climate such as air and water temperature, precipitation, humidity, snow, frost, atmosphere pressure and wind. • Environment Canada - An archive of Canadian climate normals for temperature, precipitation, wind, evaporation, sunshine and solar radiation. Archived data is available either on diskettes or CD-ROM. • Environment Canada - The Monthly Meteorological Summary (MMS): a synopsis of last month's weather data such as rain, snowfall, temperature, wind for every hour of each day of the past month. • Some municipalities - They may have local precipitation and other climatic data collected to meet local needs.

Table 7A.1 (2 of 3): Primary Data Sources - General Listing

Data Type	Data Sources and Description

Stream Flow Data	• Environment Canada in cooperation with conservation authorities -
	  • "Surface Water Data, Ontario": daily, daily mean, maximum daily, minimum daily and maximum instantaneous discharges for gauging stations throughout Ontario.*
	  • "Surface Water Data Reference Index, Canada" : data on station number, station name, drainage area, gauge location, summary of flow records, type of gauge and operation schedule for gauging stations throughout Canada.*.
	  • "Historical Stream Flow Summary, Ontario": a summary of monthly mean, annual mean, annual maximum, annual minimum, annual instantaneous and total discharges for gauging stations throughout Ontario.*.
	  • Reference Index, Hydrometric Map Supplement: a series of maps, identifying the locations of gauging stations throughout Canada at which hydrometric data is available. The maps indicate the type of data collected, i.e. whether it is flow rate, water level or sediment data, and whether a station is active or discontinued. The maps are distributed individually by each province.
	  • Electronic Data Acquisition Systems for water levels at stream flow gauging stations. Stream flow information on selected gauging stations will be supplied to individuals on request.
	* Regular publications have been discontinued since 1990/1991. Data on selected stations is sent on individual request..

Stream Water Quality Data	• Environment Canada - A database which provides water quality information on the Great Lakes.
	• MOEE - A water quality database called Provincial Water Quality Monitoring Network (PWQMN) on inland streams
	• Conservation authorities - Watershed and subwatershed management plans.
	• Environment Canada - Reference Sites for Water Quality Monitoring based on Terrestrial Ecoregions (EHD).
	• University of Toronto - Index of Biotic Integrity to Quantify Stream Quality in Southern Ontario .
	• Environment Canada -
	  • "Sediment Data, Ontario": data on instantaneous suspended sediment concentrations, daily mean suspended sediment concentration, daily suspended sediment loads and dissolved solids concentrations. It also contains a summary of maximum and minimum sampled concentrations and maximum daily suspended sediment load for rivers through out Ontario.*
	  • "Sediment Data Reference Index, Canada": a tabulation of station numbers, station names, types of operation, suspended sediment (concentrations and loads), dissolved solids, particle sizes and bed loads for rivers throughout Canada.*.
	* Regular publications have been discontinued since 1990/1991. Data on selected stations is sent on individual request..

Lake and Harbour Water Levels	• Environment Canada -
	  • "Historical Water Level Summary, Ontario" *: a summary of monthly mean, annual mean, annual maximum, annual minimum, annual instantaneous water levels for lakes and rivers in Ontario.
	  • "Water Levels - Great Lakes and Montreal Harbour"*: data on monthly mean, maximum and minimum monthly mean water levels for Lake Superior, Lake Michigan-Huron, Lake St. Clair, Lake Erie, Lake Ontario and Montreal Harbour.
	* Regular publication is discontinued since 1990/1991. Data on selected stations is sent on individual request.

Table 7A.1(3 of 3): Primary Data Sources - General Listing

Data Type	Data Sources and Description
Aquatic and Terrestrial Habitat Data	• MNR and DFO - Habitat mapping sheets and field collection records. • MOEE - A data base on contaminants in sport fish for a number of lakes and rivers. • MNR - A number of publications and maps on forest resources, wildlife and fisheries. • Federation of Ontario Naturalist - A database on wildlife. • Conservation authorities - Watershed and subwatershed management plans may contain information on green valleys, biological corridors, riparian vegetation and plants. • MNR - Aerial photographs, areas of natural and scientific interest, district land use plans, endangered species, forest resource inventory reports, historical forest resource inventory, Niagara Escarpment Plan, Ontario Land Inventory (OLI) Maps, wildlife inventory and monitoring, aquatic invertebrate data, Fish Species Distribution Data System, stream inventory data, strategic fisheries data, strategic fisheries plans, wetland evaluation database, etc. • Royal Ontario Museum (ROM)- Freshwater Fisheries of Canada, Shrubs of Ontario. • Ontario Natural Heritage Information Centre (NHIC) Peterborough - Biological and Conservation Database (BCD), Information on provincially endangered species and ecological communities compiled from a number of sources including MNR, Canadian Museum of Nature, and Committee on Status of Endangered Wildlife in Canada (COSEWIC) and non-government agencies. • Municipalities - Natural heritage systems, environmental impact studies, watershed plans, environmentally sensitive areas.

Table 7A.2 (1 of 5): Primary Data Sources - Maps

Map Type	Description	Scales	Sources
Base Maps			
Ontario Base Maps	Ontario Base Maps contain topographic information, drainage routes (streams), spot elevations, contours, boundaries, buildings, bridges, culverts, highways, roads, railways, pipelines, transmission lines, marshes and wooded areas.	1:20,000 1:10,000 1:2,000	MNR
Provincial Series Maps	Provincial Series Maps are maps featuring transportation routes, streams, political and administrative boundaries, population centres, transmission lines, dams, roads, provincial and national parks. These maps cover northern Ontario only.	1:100,000	MNR
Territorial Series Maps	They feature major drainage and transportation routes, political and administrative boundaries, population centres, transmission lines, dams, highways, roads, provincial and national parks.	1:600,000	MNR
National Topographic Systems	The maps feature political and administrative boundaries on national and provincial levels. They show lakes, rivers, streams, highways, roads, airports, canals, railways and power lines.	1:2,000,000 1:1,000,000 1:500,000 1:250,000 1:125,000 1:50,000 1:25,000	Natural Resources Canada
Municipal Boundaries (Northern Ontario)	The maps identify cities, towns, counties, regions, municipal and geographic boundaries in northern Ontario.	1:1,009,920 1:2,019,840	MMA
Municipal Boundaries (Southern Ontario)	The maps identify cities, towns, counties, regions, municipal and geographic boundaries in southern Ontario.	1:631,200 1:1,009,920	MMA
District Maps	The maps show private lands, crown lands, provincial parks, forest, fish species, recreation areas, lakes, rivers and streams.	1:125,000	MNR
Official Road Maps of Ontario	The maps feature highways, roads, administration boundaries, major population centres, travel and tourist information.	1:700,000 for Southern Part; 1:1,600,000 for Northern Part	MTO
Ontario Transportation Map Series	The maps feature highways, roads, surfacing, railways, airports, ferry routes, parks and conservation areas, car pool sites, service stations, tourist attractions, city and town enlargements.	1:250,000	MTO
County Map Series	They show lots and concessions	1:100,000	MTO

Table 7A.2 (2 of 5): Primary Data Sources - Maps

Map Type	Description	Scales	Sources
Land Use Maps			
Land Classification (Ontario Land Inventory)	The maps feature physiographic site classification, soil textures and depths. They also show highways, roads, marshes, swamps, canals, streams, railways, political and administrative boundaries.	1:250,000	MNR
Land Use Maps	Land Use Maps (Municipal) are produced in association with the preparation of watershed plans and Official Plans. The maps contain information on land development such as industrial, commercial, institutional, residential, parks, cemeteries and open spaces.	Varies	Municipalities
Agricultural Land Uses	The maps feature different field crop systems: corn, hay, grain, pasture and grazing area.. They also show idle agricultural lands, woodlands, reforestation, swamps, bogs, extraction pits and quarries.	1:50,000	OMAFRA
Climatic and Hydrologic Maps			
Floodline Maps	The maps contain floodlines and fill lines for the Regulatory storm, spot elevations and contours, watershed boundaries, political and administrative boundaries, highways, roads, rivers and tributaries, streams, bridges and culverts.	1:2,000	CA/MNR
Public Information Flood Risk Maps	The maps show normal water surfaces and designated flood risk areas, buildings, highways, roads, railways, airports and population centres.	1:25,000 1:10,000	MNR
Artificial Drainage Systems	The maps identify watershed boundaries, drains, tile drains, highways, roads, rivers, streams.	1:25,000	OMAFRA
Climatic Maps	A series of maps portraying Canada's climate such as air and water temperature, precipitation, humidity, snow, frost, atmosphere pressure, sunshine and wind.	Varies	Environment Canada
Canadian Climate Normals Atlas	This atlas consists of national maps of the 1961-1990 Canadian climate normals. The monthly and annual maps are available for mean daily maximum temperature, mean daily minimum temperature, mean daily temperature, total rainfall, total snowfall, and total precipitation.	Varies	Environment Canada

Table 7A.2 (3 of 5): Primary Data Sources - Maps

Map Type	Description	Scales	Sources
Climatic and Hydrologic Maps (Cont'd.)			
Hydrological Atlas of Canada	A series of maps providing the following hydrologic information: • Annual precipitation. • Depth, duration and frequency of point rainfall; 10 min, 60 min and 24 hour duration on 2, 5, 10 and 25 year return period. They also provide the ratios of 6 hr, 48 hr and 72 hr to those of 24 hr durations. • Annual snowfall. • Dates of formation and loss of snow cover. • Mean maximum depths of snow and time of occurrence. • Freeze-up and break-up of rivers and lakes. • Wind. • Large lakes. • River systems of Canada. • Locations of stream flow gauging stations. • Annual large river flow. • Annual runoff. • Suspended sediment concentrations. • Groundwater observation wells. • Surficial hydrogeology. • Bedrock hydrogeology.	1:10,000,000	Natural Resources Canada
Drainage Basin Hydrology	These maps are available in sets containing information on: • Surface geology and topology. • Bedrock geology and topography. • Locations of water wells. • Well yield probability. • Groundwater quality.	Varies	MOEE
Groundwater Maps			
County/District Groundwater Probability	These maps are available in sets containing information on: • Probable yields from wells. • Groundwater quality. • Well locations.	Varies	MOEE
Susceptibility of Groundwater to Contamination	The maps feature the susceptibility of groundwater to contamination based on four levels of contamination.	1:50,000	MOEE
Major Aquifers in Ontario	The maps feature locations of selected wells, depths to water zone and static water levels and water level contours.	1:100,000	MOEE

Table 7A.2 (4 of 5): Primary Data Sources - Maps

Map Type	Description	Scales	Sources
Soil and Geologic Maps			
Soil Maps	The maps show soil classifications, roads, boundaries, contours, drainage classes, topographic classes and rock classes. Soil maps provide detailed morphological, chemical and physical description of soil.	1:63,360	OMAFRA Ontario Institute of Pedology
Geological Highway Maps of Southern Ontario	They show the distribution of rock units exposed at the surface, landform features such as escarpments, canyons hills and valleys, highways and roads.	1:800,000	MNR
Physiography of Southern Ontario	They show the surficial geological features such as fill moraine, till plains and spillways. Township, county, district and regional boundaries are also shown.	1:250,000	MNR
Data Series Preliminary Maps	They show geological data, rock types, mining areas and data such as drill holes, shaft depths, trenching, etc.	1:15,840	MNR MNDM
General Construction Capability Map (Northern Ontario Engineering Geology Terrain Study)	The maps show regional engineering terrain conditions, suitability of various terrains for general construction activities.	1:100,000	MNDM
Precambrian Geology	The maps identify mineral and metal occurrences, rock types and boundaries, bedrock outcrops, mining properties, district, regional, municipal and geographic township boundaries, population centres, railways, highways and roads.	Varies	MNDM
Quaternary Geological Publications	They show bedrock contours, bore holes location, bedrock outcrops, highways and roads, rivers and lakes.	1:50,000 1:2,880	MNDM
Toronto-Windsor Area Geology (Geological Survey of Canada)	The maps feature topographic contours, palaeozoic geology, geological boundaries, quarries, rock outcrops, faults, population centres, highways, roads, railways, rivers and creeks.	1:250,000	Natural Resources Canada

Table 7A.2 (5 of 5): Primary Data Sources - Maps

Map Type	Description	Scales	Sources
Fisheries and Environmental Data			
Fisheries Maps	The maps show lake characteristics, locations, access, water depths, temperature and fish species.	Varies	MNR
Trout and Salmon Migratory Routes	The maps show former, present and potential migration routes for 9 trout and salmon species. They also show details of 187 streams in Ontario.	1:760,320 Southern Ontario 1:1,009,920 Northern Ontario	MNR
Land Capability for Forestry (Canada Land Inventory)	They show different mineral and soil groups. They are grouped into one of seven classes based on their inherent ability to grow commercial timber. Productivity species is also shown.	1:250,000	Natural Resources Canada
Land Capability for Agriculture (Canada Land Inventory)	They show different mineral and soil groups. They are grouped into one of seven classes based on intensity and limitations for agriculture.	1:250,000	Natural Resources Canada
Land Capability for Wildlife - Ungulates (Canada land Inventory)	The maps feature different land which is classified into 7 classes based on the physiographic characteristics (such as vegetation, climate and ecology) important to wild ungulates.	1:250,000	Natural Resources Canada
Land Capability for Wildlife - Waterfowl (Canada land Inventory)	The maps feature different land which is classified into 7 classes based on the physiographic characteristics (such as vegetation, climate and ecology) important to wild waterfowl population.	1:250,000	Natural Resources Canada

Appendix 7B
Practical Aspects of Field Investigation

Disclaimer

Health and safety must always receive the utmost attention in a field investigation and must never be compromised. Although this appendix discusses the health and safety aspects of field investigation, the discussions must not be regarded as official procedures, nor that they cover all matters and situations, nor that they are up-to-date. It remains the sole responsibility of the office and supervisor to comply with all the health and safety legislative requirements and the policies and procedures of the organization which undertakes the field investigation.

Practical Aspects (Field Trip Aid)

Typical practical aspects of field investigation include the following:

- Health and safety
- Forest fire prevention
- Travel and meeting arrangement
- Prepare checklist and investigation forms
- Permission to enter private land
- Accessibility of site
- Remote area investigation
- Winter investigation
- Field equipment
- Local information

Health and Safety

The Occupational Health and Safety Act is a major piece of legislation governing the health and safety of workers. The Act explains the responsibilities of employers and employees, and their respective duties of care. Supervisors, who are responsible for carrying out field investigations, should be familiar with current health and safety practices. A field trip should not be undertaken by a single person.

Staff in the field must exercise due care and common sense in addition to complying with current safety practices. It is extremely important that site trips are planned thoroughly, particularly in remote areas and under adverse conditions. The person in charge should attempt to anticipate hazardous situations and consider alternatives to reduce or eliminate hazards wherever possible. In hazardous situations, such as during a severe flood, investigations should be postponed until conditions improved.

There are many possible hazards to be encountered on a field trip. The following lists mention some typical hazards but other unmentioned hazards may exist.

- Health Hazards
  - Insect bites and stings (Use loose, light-coloured clothing. Use repellant against blackflies.)
  - Frostbite and hypothermia (Cause of most weather-related deaths, even in above-freezing temperatures.)
  - Animal bites such as dog and snake bites (Avoid animals behaving abnormally.)
  - Cuts, sprained or broken limbs, muscle injury etc.
  - Polluted water (drink only treated water and avoid body contact)
  - Heat stroke and exhaustion.

- Safety Hazards
  - Water - drowning.
  - Falling.
  - Head injury.
  - Foot injury.
  - Breaking through ice on rivers or lakes.
  - Traffic.
  - Poisonous vapours from sewers.
  - Flooding and flash storms during sewer inspections.

Forest Fire Prevention

Extreme care must be taken to prevent forest fires while travelling in woodlands. Rules and regulations set down in the Forest Fires Prevention Act must be carefully observed. No person should smoke while walking or working in a forest or woodland during a fire season. This is normally from April 1 to October 30, but may be extended by MNR.

Fire for warmth or cooking should be done on bare rock and free from flammable material. When the forest fire hazard is extremely high, MNR may designate an area to be a restricted fire zone during which off road travel require a permit. Field investigators are advised to consult MNR for up-to-date information.

Travel and Meeting Arrangement

The itinerary and timetable should be determined before the trip. It is important to select the most economical route. If there is more than one site, the order to inspect each site should be planned.

The length of time required should be planned as closely as possible, based on the estimated time per site visit. The time spent on an investigation should bear some relation to the size, importance and complexity of the project.

Site meetings should be arranged well in advance especially when a large group of people is involved.

If a special mode of transportation is required, for example, a boat, it should be arranged well in advance of the trip.

Prepare Checklist and Investigation Forms

It is important to prepare a check list to identify all the important features to be inspected. Investigation forms could also be used as reminders.

When a checklist or investigation form is used, fill in as much data as possible from office records. This will reduce the time spent on site. It is important that the pre-filled data be checked and verified on site as data is sometimes incorrect due to changes. For example, culvert dimensions and bed elevations should be verified by site measurement.

Some typical investigation forms which may be used are enclosed at the back of this appendix.

Permission to Enter Private Land

Under normal circumstances, permission to gain access to private land should be obtained from both the owner and occupant. Although Section 6 of *the Public Transportation and Highway Improvement Act* allows, under certain circumstances, entering into any private land without the consent of the owner, employees must make every reasonable effort to obtain permission to enter.

Accessibility of Site

For every field trip, it is very important to check the route to the study area. This is especially the case when the site is in a remote location. For a remote site that has only one vehicle access route, it is extremely important to make sure that the route is still accessible before the start of the journey. For a site that is only accessible by foot, find the nearest safe location that you can leave the car. If the remote area is only accessible by boat, allow plenty of time for a boat trip.

Remote Areas Investigation

The local MNR offices or other appropriate persons should be informed of details of proposed trips into remote areas. Arrange for emergency support as necessary. A compass must always be carried in bush areas, together with a survival and first aid kit.

Winter Investigation

Investigations under winter conditions are often unsatisfactory because important data may be concealed by ice and snow. Investigations during winter months should be avoided where possible by adjusting the scheduling of projects. The information gathered should be confirmed and augmented by a further inspection after the spring flood has receded.

Field Equipment

The following is a general checklist of field equipment. It is not to be considered an exhaustive list.

- Health and Safety Equipment.
  - Hard hat where required.
  - Safety vest where required.
  - Insect repellant.
  - Sunscreen protective Lotion.
  - Rainwear where required.
  - Lifejacket.
  - Survival gear for remote area travel such as compass, waterproof matches, first aid kit, food, water etc.
  - Appropriate winter clothing and equipment.
  - Nylon ropes.
  - Safety footwear.
  - Two-way radio.
  - Traffic safety equipment
  - Other safety and rescue equipment when appropriate.

- Documents and Data.
  - Topographic maps (appropriate scale).
  - Aerial photos (enlargements if required).
  - Standard field forms.
  - Survey plans and profiles.
  - Road maps and city maps.
  - Identification and business cards.

- Site information such as flood dates, HWL data, field notes of previous projects in the vicinity.
- Miscellaneous information such as addresses, phone numbers of officers and people to be visited.

- Stationery and sampling equipment as applicable

  - Pencils, grease pencils.
  - Long measuring tape (30 m), short measuring tape (5 m).
  - Weight with clip for attaching to loop of long tape.
  - Range pole (s).
  - Camera and film.
  - Hand level.
  - Pocket stereoscope.
  - Current meter and accessories.
  - Stopwatch.
  - Boat, motor and accessories.

Local Information

Some typical sources are:

- Highway maintenance staff - They are particularly helpful for providing flood levels on small culverts, which are not often noticed by the general public.
- Local residents - They are often the best source of information. Farmers living adjacent to a stream crossing generally provide useful and reliable information, especially when their property has been flooded in the past. Long term residents ma possess more information than do short term residents. Be aware that residents may tend to exaggerate past flooding conditions or may have other concerns that may affect their responses to queries. Information should be obtained from at least two sources to provide some means of cross reference.
- Conservation authorities - In addition to flood plain and flood hazard maps, they often have useful records and reports on past floods. Sometimes they have profiles and airphotos of past floods.
- Ministry of Natural Resources - MNR staff have knowledge of local streams. They sometimes have airphotos of floods in progress.
- Ontario Hydro - They can provide relevant flood levels in rivers where a dam is operating.
- Local Newspaper - Local newspapers may provide a valuable source of historical flood events. They contain a diversity of information such as flood details, photographs, flooding problems, degree of severity, ice jams, etc. It is essential to know the approximate date of a flood before searching. The approximate date could be obtained from local residents or from stream flow records. Very often, local libraries have rolls of microfilms containing newspaper archives. In view of the time requirement and high level of effort, this type of information should be searched only when it is justifiable.
- Other sources - These include municipal officials, consultants, railway maintenance staff, police stations, etc.

Table 7B.1: Local Information Record Form

1	Site referred to	
2	Informant's name/phone no.	
3	Location relative to site	
4	Period of knowledge	
5	Max. H.W.L. - Date	
	- Details	
6	2nd H.W.L. - Date	
	- Details	
7	Average annual H.W.L.	
8	Relief flow (describe)	
9	Past washouts of road	
10	Past raising of road	
11	Other by-passing flow	
12	Ice or debris jamming	
13	Past struct. performance	
14	Misc. information:	
	a Present flow condition	
	b Channel dredging	
	c Municipal Drain?	
	d Flooding of property	
	e Upstream dam washouts	
	f Erosion problems	
	g Beaver problems	
	h Navigation	
	i Subsoil information	
	j Fish resources	
	k Flood control plans	
	l Other information	
15	Photos of floods etc.	
16	Names, addresses and telephone numbers of people/officials to be interviewed	

Table 7B.2: Field Investigation Form for Major Stream Crossing

Name of Stream ______________________ W.P. No. __________

Location ______________________

Road Name/No. __________ Site No. __________ File No. __________

Owner: Prov./Mun. __________ Insp. By __________ Date __________

Topog. Map No. __________ Airphoto Nos. __________

Flow Conditions at Time of Inspection ______________________

SPECIAL FEATURES OF STREAM

1 Channel type/stability ______________________
2 Degradation/aggradation ______________________
3 Dredging (past or fut.) ______________________
4 Ice data ______________________
5 Debris data ______________________
6 Beaver activity ______________________
7 Logging activity ______________________
8 Navigation ______________________
9 Existing dams ______________________
10 Misc. stream controls ______________________
11 Environmental data ______________________
12 Special watershed data ______________________
13 Other information ______________________

Table 7B.3: Field Investigation Form for Minor Stream Crossing

Reg./Dist. ____________________ Hwy. No. __________ W.P. No. __________

Station ____________________ Insp. By __________ Date __________

Name of Stream/Drain ____________________

Flow Conditions at Time of Insp. ____________________

SPECIAL FEATURES AND PROPOSED CROSSING

1 High-water levels ____________________

2 Natural bed material ____________________

3 Potential problems ____________________

4 Fisheries potential ____________________

5 Properties affected ____________________

6 Manning's 'n' data ____________________

7 Other information ____________________

8 Provide sketch of proposed crossing on reverse side of form.

EXISTING CULVERTS (At site and upstream/downstream crossings)

9 Location			
10 Description/age			
11 Dimensions			
12 High-water levels			
13 Relief flow			
14 Bed/invert mat'l			
15 Past problems			
16 Other information			
17 Est. adequacy			

LOCAL INFORMATION

18 Site referred to ____________________

19 Details of informant ____________________

20 Details of floods ____________________

____

21 Relief flow ____________________

____

22 Ice/debris problems ____________________

23 Culvert performance ____________________

24 Misc. information ____________________

____

____

____

____

Table 7B.4: Field Investigation Form for Existing Structure

1 Location & site no. ______________________

2 Descrip. & approx. age ______________________

3 High-water levels ______________________

4 Relief flow ______________________

5 Past raising of grade ______________________

6 Nat. stream bed mat'l. ______________________

7 Invert mat'l. (if diff.) ______________________

8 Scour damage/repairs ______________________

9 Degradation/dredging ______________________

10 Other information ______________________

11 Estimated adequacy ______________________

CHECKLIST OF DIMENSIONS REQUIRED ON SKETCH

12 ☐ Span length(s) & skew angle

13 ☐ Height: soffit/crown to road grade

14 ☐ Height: soffit/crown to WL

15 ☐ Depth of sag on approach grade

16 ☐ Depths: WL to streambed (invert)

17 ☐ Depths: soft material below bed

18 ☐ Average depth from WL to bed (away from structure) ______________________

ELEVATION VIEW: ☐ UPSTREAM OR ☐ DOWNSTREAM (Show plan view on reverse side if req'd.)

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Purpose of This Chapter

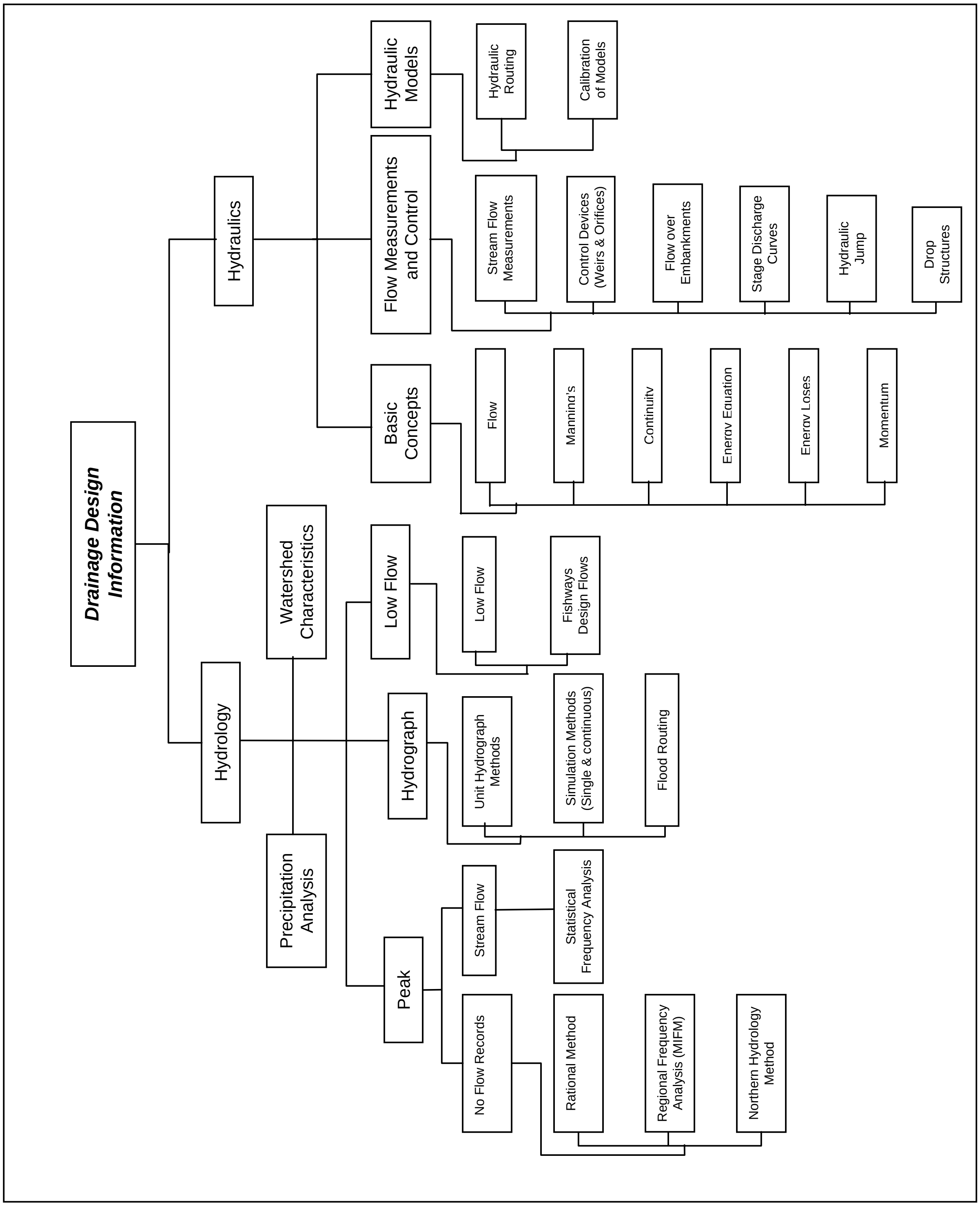
The purpose of this chapter is to provide the theoretical concepts and principles of hydrology, hydraulics and water quality management that designers require to develop drainage system designs.

In this chapter, information on hydrology is focused on the estimation of floods for both large and small catchment areas. The hydraulic information provided deals with the measurement and movement of floods through highway drainage systems.

This chapter can be used as a reference in determining the accepted computational procedures and methods, applied in parts I and II of this manual, so that the design and analysis of highway drainage systems would be consistent and scientifically sound. Examples are provided to illustrate the application of theories, formulae and procedures. Designers will be able to assess what the methods can accomplish and their limitations in application.

The hydrology and hydraulics information covered in this chapter is organized in an order that corresponds to the order of computation methods that a user will have to follow when developing a design, as illustrated in Figure 8.1. This figure also provides an easy reference to the information contained in this chapter.

Figure 8.1: Computation Selection Procedure



Precipitation Analysis

Precipitation occurs in the form of rain and snow and is the major source of runoff and stream flow. Without knowledge of the amount and distribution over time and space of precipitation, an assessment of the runoff potential of a drainage area cannot be made accurately.

This section provides the information that is needed to make the assessment. It further presents a discussion of the various storms and their distributions so that the best and most appropriate one can be selected, in a format required by most hydrologic simulation models.

The Hydrologic Cycle

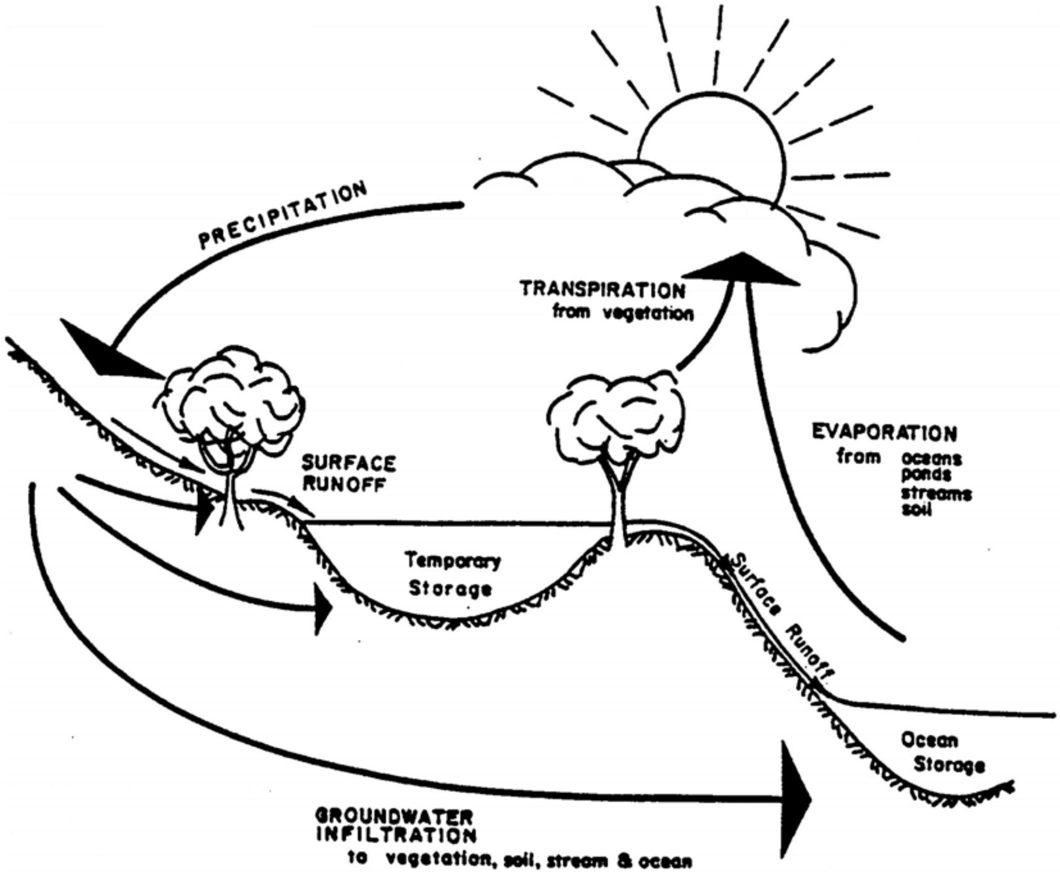
The hydrologic cycle deals with the occurrence, distribution, and movement of water, and the inter-relationship between these factors. The hydrologic cycle is illustrated in Figure 8.2 which highlights the various processes associated with the movement of water in nature. Understanding these processes is important for the development and application of hydrologic models.

As shown in Figure 8.2, following the occurrence of a precipitation event, some of the precipitation returns to the atmosphere through evaporation and transpiration. A portion of the precipitation infiltrates into the soil and eventually reaches a nearby stream or percolates and recharges the groundwater supply. The remaining water flows overland to streams and lakes. Eventually, both surface and groundwater reach the ocean.

The hydrologic process has been simplified to present a summary of the various physical processes affecting the development of a drainage design. Very often a drainage plan, especially in an urban environment, has the potential to change the hydrologic cycle. The drainage plan may require modifications to a pre-development environment that include changes such as, alteration of the natural drainage paths, damming of waterways for flood control, creation of reservoirs, and implementation of efficient stormwater management systems like sewers and concrete lined channels. Such changes can significantly modify the hydrologic balance due to, for example, the reduction in infiltration resulting in diminished groundwater recharge and consequently higher surface runoff volumes and peak flow rates.

It should be recognized that, as water cycles through the various processes, its quality changes. Some process such as infiltration remove impurities, while others, like movement through urban areas, are likely to cause degradation to the water quality.

Figure 8.2: The Hydrologic Cycle



(MTO, 1986)

Design Rainfall

Modern methods of computation have greatly enhanced the possibility of using the actual rainfall data over several years in evaluating the runoff potential of a drainage area. However, this is quite infrequently done due to the cost and the time it takes for the analysis to be done. Instead, a representative design storm is adopted. As a result, several flow estimation models have been developed that utilize only a design storm.

In developing a design storm, a representative rainfall pattern, type and distribution should be identified.

In this section, the different rainfall types and their distributions which are recommended for developing MTO drainage systems are further discussed.

Characteristics of a Design Storm

The amount of rainfall and its distribution in time and space are usually critical inputs in calculating runoff for drainage system design. However, depending on the type of analysis or the design that is undertaken, other characteristics of the rainfall may be considered. Some of these may include the following:

- instantaneous and average rainfall intensity;
- maximum or minimum volume over a given duration;
- total volume (depth) and storm duration;
- net precipitation;
- statistical distributions and classification of rainfall;
- storm type and specific applications; and
- areal and temporal distribution.

Available Data Types

It is important that some investigation of the various sources of rainfall data be undertaken so that the most appropriate sources can be identified. In this section the various sources are identified and discussed so that designers may become more selective in choosing a particular data type. These sources include of:

- intensity duration frequency (IDF) curves;
- historical storms (actual rainfall measurements); and
- representative design storms, typically Soil Conservation Service (SCS) and Chicago storms;

These data types are discussed more fully in subsequent sections.

Intensity Duration Frequency (IDF) Curves

An IDF curve is a statistical relationship of rainfall intensity corresponding to a specified storm duration and frequency for a given location.

IDF curves are used where precipitation field measurements on a drainage system are not available, and precipitation must be estimated from secondary sources. Besides being quick and easy to use, this data are readily available. IDF curves are one of the most widely used forms of rainfall data.

IDF rainfall data is used to estimate flows for the design of minor drainage systems with the Rational method. With IDF curves, it is possible to determine the average rainfall intensity likely to be attained or exceeded in a specific location and for a specific frequency at a given location. IDF curves are also used in the development of synthetic storm distributions for watershed routing analysis and modeling. Figure 8.3 shows a typical IDF curve.

Derivation of the IDF Curve

IDF curves are statistically derived from rainfall records compiled over a period of years. IDF curves are usually derived using an annual duration series made up of the largest rainfall intensities recorded in any one calendar year. Lesser values in the same year are ignored, even though they may exceed some maximum values for other years.

Numerically, IDF curves are expressed in an equation of the form:

$$i = A / (t_d + B)^c \qquad (8.1)$$

where:

i = average rainfall intensity, mm/h
t_d = rainfall duration, min
A, B, and c are coefficients.

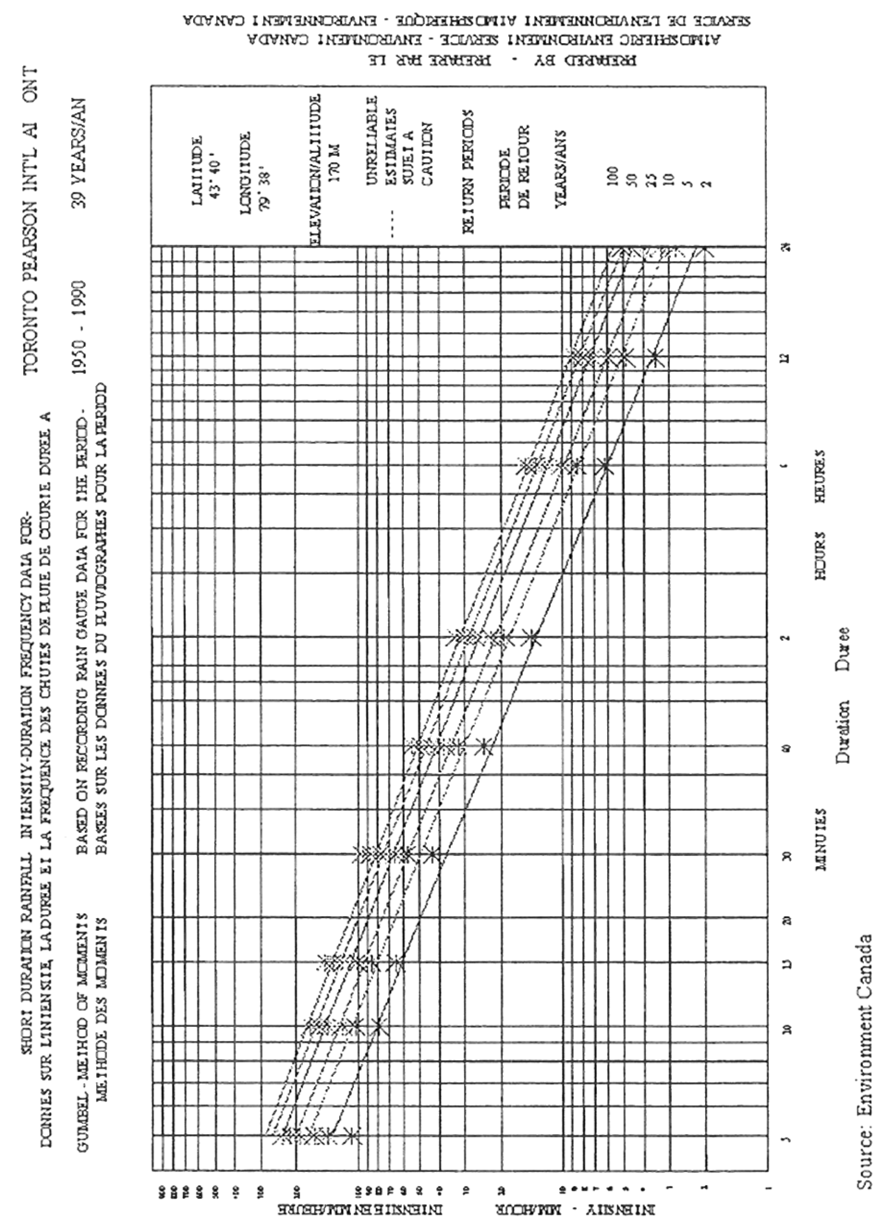
Using a logarithmic linear transformation of Equation 8.1 to derive:

$$\text{Log } i = \log A - c \log (t_d + B) \qquad (8.2)$$

Equation 8.2 has the form of a straight line, where c represents the slope. This equation can be solved by the least squares statistical method.

The coefficient B is an unknown value which must be solved by trial and error. In doing so, a first value is assumed and the regression analysis is used to determine the values for A and c. The process is repeated for other values of B until the correlation coefficient closest to -1 is determined. The calculation of the coefficient A, B and c are shown in Example 8.1.

Figure 8.3: Typical Intensity-Duration-Frequency (IDF) Curve



Accuracy of IDF Curves

The accuracy of rainfall intensities estimated by the IDF method depends greatly on the length of records at the station relative to the desirable return period. If the length of record is less than the desirable storm event return period, the accuracy will be relatively lower. Where the accuracy is lower, IDF curves for a more distant station having a longer period of record may be used, provided that the climatic conditions are similar.

The extrapolation of frequency curves for return periods greater than twice the length of record is not recommended.

As noted earlier, IDF values describe the variation of extreme point rainfall with time for a given frequency at a given station. The values do not account for the variation with area. Therefore, areal reduction factor should be applied to all IDF values. A more detailed discussion of areal reduction is presented in the section on Areal Adjustment of Rainfall in this chapter.

Sources of IDF Data

The sources of IDF curve data include:

- IDF map of Ontario;
- MTO District IDF curves; and
- Atmospheric Environment Service (AES) data.

IDF Map of Ontario

The IDF map of the Province of Ontario is also available from the publication *Rainfall Frequency Atlas of Canada* (AES, 1985). The map is particularly useful in indicating rainfall patterns, rather than information for design purposes.

The publication also shows contours of mean annual rainfall extremes (isohyets) and standard deviations of the rainfall intensities for durations ranging from 5 minutes to 24 hours.

MTO District IDF Curves

Information from the isohyets on the maps presented in the *Rainfall Frequency Atlas of Canada* has been used to develop regional IDF values for each MTO districts.

The MTO district IDF tables can be used in the design and analysis of highway drainage systems. However, the IDF values represent average values for each district. It is advisable that the analyst/designer compares the district curve values to the AES values. The IDF data for MTO districts are given in Design Charts 1.01(a) to 1.01(t).

Atmospheric Environment Service (AES) Data

The Atmospheric Environment Service (AES) of Environment Canada operates and maintains a large network of precipitation gauges in Ontario.

For stations with more than 10 years of data, IDF curves are statistically derived. They are calculated using a method developed by Gumbel, which calculates the rainfall intensity with a given return period according to the following equations:

$$x(t) = \underline{x}\ K(T) \tag{8.3}$$

$$K(T) = \frac{-\sqrt{6}}{\pi} + \left(\ln\left[\ln\left[\frac{T}{T-1}\right]\right]\right) \tag{8.4}$$

where:

$\underline{x}$	= average of x values		
t	= intensity at time t		
T	= return period		
K(T)	= frequency factor	= -0.164 for 2-yr storm;	= 2.044 for 25-yr storm;
		= 0.719 for 5-yr storm;	= 2.592 for 50-yr storm;
		= 1.305 for 10-yr storm;	= 3.137 for 100-yr storm.

IDF information is very useful for design purposes as the data often relate to local gauges within a project area.

Storm Duration

The duration of a storm varies with the type of analysis. Watersheds with storage facilities may require the use of a long duration storm such as 12 or 24 hours. For small urban watersheds, a 1 to 6 hours duration for rainfall-runoff simulation may be appropriate. However, where there is a distinct possibility that the highway system may be impacted if there is failure of the drainage system several storm durations should be simulated and the most critical one chosen.

A storm duration of 24 hours is more widely accepted for large drainage basins or basins with ponds. Storm sewers may be sized using shorter duration storms. However, it is the designers' responsibility to recommend and justify a storm duration.

It is recommended that the storm duration should at least be equal to the time of concentration, and is usually taken as 3 or 6 hours or a multiple thereof.

Areal Adjustment of Rainfall

For areas larger than 25 km^2, the point rainfall derived from the IDF curves should be reduced by applying an areal adjustment factor obtained from Design Chart 1.06 or other acceptable sources. This is to account for reduced average rainfall intensities occurring over larger areas.

Historical Storms

A historical storm represents a flood of a magnitude which exceeds all predated events. The two historical design storms used in Ontario are Hurricane Hazel, which occurred in October, 1954, and the Timmins Storm, which occurred in September, 1961. Hazel and Timmins have been designated as the provincial regulatory storms for areas in Zones 1 and 3, respectively within Ontario. The spatial extent of the zones is shown in Design Chart 1.02. Note that Zone 1 covers Southern Ontario, while Zone 3 covers Northern and South-Central Ontario.

The Ministry of Natural Resources and conservation authorities have advised that Hurricane Hazel and the Timmins Storm should be used as the regulatory storm for drainage basins greater than 130 hectares, unless an alternative storm is authorized.

When applying a historical storm to a watershed, it is necessary to consider the antecedent soil moisture condition (AMC) of the watershed. When applying the 48-hour Hurricane Hazel (Zone 1 storm) and the 12-hour. Timmins Storm (Zone 3 storm), the average AMC II condition applies, and no adjustments are necessary. It is possible, however, to apply a 12-hour Hurricane Hazel distribution, which represents the last 12-hour of the 48-hour storm. In this case AMC III must be used to account for the previous 36 hours of precipitation. Further discussion on AMC is presented in the section on Soil Moisture.

For both regulatory storms, an areal reduction factor is applied to account for areas greater than 25 km^2. Design Charts 1.03 and 1.04 give the design rainfall distribution and the areal reduction factors. Refer to the Ministry of Natural Resources document *Flood Plain Management in Ontario, Technical Guidelines* (MNR, 1975), for more information on the application of historic storms.

Representative Design Storm

Ideally, actual storms should be used in a hydrologic analysis. However, actual storm records are not available for every location in the province. Therefore, other methods of estimating a storm need to be used. One of those methods uses statistically derived storm distributions.

Two statistically derived storms have been traditionally adopted as representative storm, namely the SCS Type II and Chicago Distributions.

SCS Type II Distribution

These distributions were developed by the Soil Conservation Service of America in 1973 to estimate floods for rural areas. They are suitable and have been adopted in Southern Ontario as one of the design storms.

The 24-hour SCS storm is generally applicable to undeveloped or rural basins (low percentage of impervious area) where peak flow rates are largely influenced by the total depth of rainfall. The design distribution is given in Design Chart 1.05.

The time step is normally the time interval chosen to represent the design storm distribution (hyetograph) in the runoff model. This should be chosen very carefully as it is related to the computational time step, and therefore the accuracy of the generated hydrograph. Small urban areas may require 10 minute time steps, whereas larger rural watersheds may require 15 to 60 minutes. A small time step (1 or 2 minutes) may result in unrealistically high flows when used with these distributions.

Chicago Distribution (Keifer and Chu Storm)

The Chicago storm was developed by C. J. Keifer and H.H. Chu based on 25 years of rainfall record for the city of Chicago. This method was published in the Journal of the American Society of Civil Engineers (August, 1957). The storm is generally applied to urban basins (high percentage of impervious area) where peak runoff rates are largely influenced by peak rainfall intensities.

The 4, 6, or 12-hour Chicago storm may be applied in the design of highway drainage facilities depending on the potential impacts and size of drainage area. However, the 24-hour should be applied for the design of storage facilities. Some guidance in selecting the storm duration is given in the section on storm duration. Figure 8.4 shows the storm distribution and defines some of the terms used in developing the storm. The procedure for the development of the Chicago storm is presented in Example 8.1.

Example 8.1: Calculation of a Chicago Storm Distribution

Required

Develop a Keifer & Chu (Chicago) storm distribution (hyetograph) for a 100-year return period event for District 11, Huntsville.

Solution

The following steps may be used to determine a 24-hour Keifer & Chu Design Storm:

1. Determine an equation which describes the IDF curve for District 11.

Use the following equation to define intensity as a function of time:

$$i = \frac{A}{(t_d + B)^c} \qquad (8.1)$$

where:

i = rainfall intensity, mm/h

t_d = duration, min

A, B, c are constants

By taking the logarithm of Equation 8.1, the equation takes the form of a straight line.

$$\text{Log } i = \log A - c \log (t_d + B) \qquad (8.2)$$

where:

c = the slope of the best fit line of the plot of, log [i] versus lLog [t_d+B] , the logarithm of intensity versus the logarithm of duration, plus a constant, B;

B = an unknown value which must be solved for by trial and error;

A = the intercept of that line.

From Equation 8.2 the constants A, B and c are obtained by fitting the intensity-duration data to the equation using linear regression.

A value for B is first assumed and the regression analysis determines values for A and c. Another value is selected and the analysis is repeated. The process continues until the correlation coefficient closest to -1 has been determined. An example is shown in Table 8.1.

Table 8.1: Regression Analysis of The IDF Curve

District 11 Huntsville Intensity (mm/h)	Duration (min)	log [Intensity] (mm/h)	log [Duration + B] (min)
215	5	2.3324	log [5 + B]
150	10	2.1761	log [10 + B]
125	15	2.0969	log [15 + B]
82	30	1.9138	log [30 + B]
55	60	1.7404	log [60 + B]
28	120	1.4472	log [120 + B]
12	360	1.0792	log [360 + B]
6.6	720	0.8195	log [720 + B]
3.7	1440	0.5682	log [1440 + B]

Regression Analysis

Trial No.	Assume B	Log A	c	Correlation Coefficient
1	7.0	3.2349	-0.8424	- 0.999 560
2	8.0	3.2691	-0.8547	- 0.999 593 ←
3	9.0	3.3019	-0.8666	- 0.999 567
4	7.9	3.2657	-0.8535	- 0.999 592
5	8.1	3.2724	-0.8559	- 0.999 592

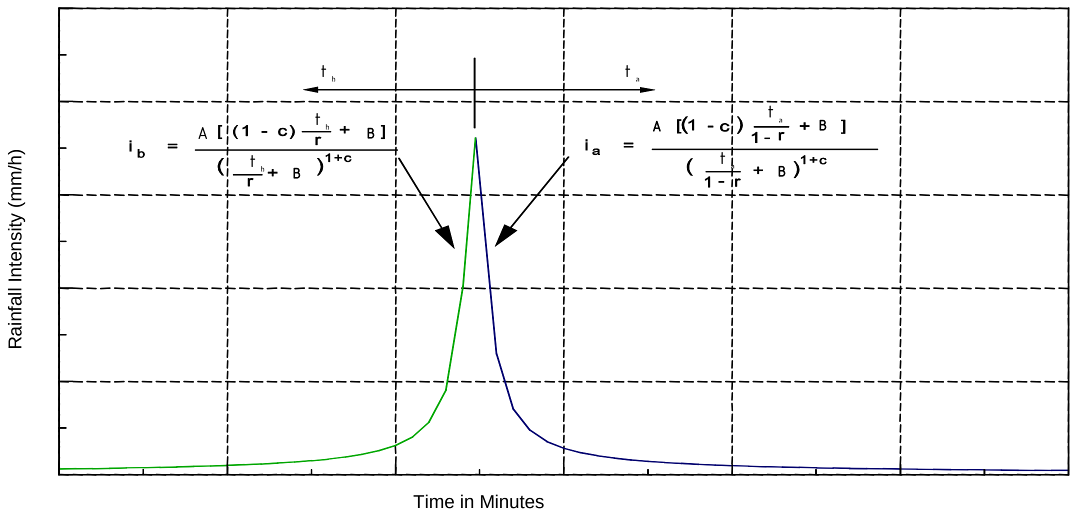
The correlation coefficient closest to -1 is - 0.999593. Therefore:

log A = 3.2691, A = 1860
B = 8.0
c = 0.8547

Intensity Equation

$$i_{100} = \frac{1860}{(t_d + 8.0)^{0.8547}}$$

Figure 8.4: Chicago Storm Distribution



2. Determine the value of r

r represents the ratio of the time of peak intensity divided by the storm duration.

The value of r is usually derived from existing rainfall records. The method involves calculating the mean values of mass antecedent rainfall and the mean location of the peaks for various rainfall durations for a series of excessive rainfall events. The process is lengthy and subject to interpretation.

An r value of 0.38 may be used for all MTO districts. This will provide a consistent application across the province.

3. Discretize the design storm

This procedure involves breaking the design storm into small time steps. A time step of 10 minutes was chosen. The value of the time step is dependent on the application and will vary for each project. The peak intensity for this type of design storm is computed using the following equation.

$$i_p = \frac{A}{(\Delta t + B)^c} = \text{peak rainfall intensity} \qquad (8.7)$$

The 10-minute intensities are then distributed around the peak as r Δ_t before the peak and (1-r) after the peak. Additional points are then computed before and after the peak, until the intensities describe all increments within the storm duration. To determine intensities before the peak, the integral form of the design storm is used.

Using the values determined for A, B, c and r, the design storm can be described using the following equations.

Before the peak:

$$i_b = \frac{A[((1-c)t_b/r) + B]}{[t_b/r + B]^{1+c}} \quad (8.5)$$

$$\int_{t_{b1}}^{t_{b2}} i_b dt_b = \left[\frac{At_b}{([t_b/r] + B)^c} \right]_{t_{b1}}^{t_{b2}} \quad (8.8)$$

After the peak:

$$i_a = \frac{A[((1-c)t_a/(1-r)) + B]}{[t_a/(1-r) + B]^{1+c}} \quad (8.6)$$

$$\int_{t_{a1}}^{t_{a2}} i_a dt_a = \left[\frac{At_b}{([t_a/(1-r)] + B)^c} \right]_{t_{a1}}^{t_{a2}} \quad (8.9)$$

A simple spreadsheet is developed to discretize the District 11 design storm and the results are shown on Table 8.2.

Table 8.2: Storm Discretization

A = 1860
B = 8.0
c = 0.8547
r = 0.38
t_d = 10 min

t_d * r = 3.8
t_d * (1 - r) = 6.2
i_p = peak rainfall intensity, mm/h
t_b = time before the peak intensity, min
t_a = time after the peak intensity, min

$$i_p = \frac{1890}{(10 + 8.0)^{0.8547}} = 154 \text{ mm/h (storm time = 550 min)}$$

t_b (min)	Storm Time (min)	$i_b\ dt_b$	$A(i_b\ dt_b)$	Storm Intensity $\frac{A(i_b dt_b)}{t_d}$ (mm/h)	t_a (min)	Storm Time (min)	$i_a\ dt_a$	$A(i_a\ dt_a)$	Storm Intensity $\frac{A(i_a dt_a)}{t_a}$ (mm/h)
3.8		589.1			6.2		981.8		
13.8	540	993.2	404.3	40.4	16.2	560	1482	500	50.0
23.8	530	1149.9	156.6	15.7	26.2	570	1722	240	24.0
33.8	520	1243.3	95.7	9.6	36.2	580	1876	154	15.4
43.8	510	1314.4	68.8	6.9	46.2	590	1988	112	11.2
53.8	500	1368.3	53.9	5.4	56.2	600	2076	88.1	8.8
63.8	490	1412.6	44.4	4.4	66.2	610	2149	72.6	7.3
73.8	480	1450.4	37.8	3.8	76.2	620	2210	61.7	6.2
83.8	470	1483.3	33.0	3.3	86.2	630	2264	53.7	5.4
93.8	460	1512.6	29.3	2.9	96.2	640	2312	47.6	4.8
103.8	450	1539.0	26.4	2.6	106.0	650	2354	42.8	1.3
113.8	440	1563.0	24.0	2.4	116.0	660	2393	38.9	3.9
123.8	430	1585.1	22.1	2.2	126.0	670	2429	35.6	3.5
133.8	420	1605.5	20.4	2.0	136.0	680	2462	32.9	3.3
143.8	410	1624.5	19.0	1.9	146.0	690	2492	30.6	3.1
153.8	400	1642.3	17.8	1.8	156.0	700	2521	28.6	2.9
163.8	390	1659.1	16.8	1.7	166.0	710	2548	26.6	2.7

Table 8.2: Storm Discretization (Continued)

t_b (min)	Storm Time (min)	$i_b\ dt_b$	$A(i_b\ dt_b)$	Storm Intensity $\frac{A(i_b dt_b)}{t_d}$ (mm/h)	t_a (min)	Storm Time (min)	$i_a\ dt_a$	$A(i_a\ dt_a)$	Storm Intensity $\frac{A(i_a dt_a)}{t_a}$ (mm/h)
173.8	380	1674.9	15.8	1.6	176.0	720	2573	25.4	2.5
183.8	370	1689.9	15.0	1.5	186.0	730	2597	24.0	2.4
193.8	360	1704.2	14.3	1.4	196.0	740	2620	22.8	2.3
203.8	350	1717.8	13.6	1.4	206.0	750	2642	21.7	2.2
213.8	340	1730.8	13.0	1.3	216.0	760	2663	20.7	2.1
223.8	330	1743.2	12.5	1.2	226.0	770	2682	19.8	2
233.8	320	1755.2	12.0	1.2	236.0	780	2701	19.0	1.9
243.8	310	1766.7	11.5	1.1	246.0	790	2720	18.3	1.8
253.8	300	1777.8	11.1	1.1	256.0	800	2737	17.6	1.8
263.8	290	1788.5	10.7	1.1	266.0	810	2754	17.0	1.7
273.8	280	1798.8	10.3	1.0	276.0	820	2771	16.4	1.6
283.8	270	1808.6	10.0	1.0	286.0	830	2786	15.8	1.6
293.8	260	1818.4	9.7	1.0	296.0	840	2802	15.3	1.5
303.8	250	1827.8	9.4	0.9	306.0	850	2817	14.8	1.5
313.8	240	1836.9	9.1	0.9	316.0	860	2831	14.4	1.4
323.8	230	1845.8	8.9	0.9	326.0	870	2845	14.0	1.4
333.8	220	1854.4	8.6	0.9	336.0	880	2859	13.6	1.4
343.8	210	1862.8	8.4	0.8	346.0	890	2872	13.2	1.3
353.8	200	1871.0	8.2	0.8	356.0	900	2885	12.9	1.3
363.8	190	1878.9	7.8	0.8	366.0	910	2897	12.6	1.3
....	....	....	......	......	....	....	....	....	....

Watershed Characteristics Affecting Runoff

To calculate the runoff generated from a watershed it is necessary to know the hydrologic response of the watershed to precipitation. The hydrologic response of a watershed (outflow characteristics) is dependent on the physical characteristics of the watershed such as slope, basin shape, size and topography. Estimating flow from a watershed can be sensitive to some, or all of these characteristics. Therefore, poor assessment of these characteristics can lead to significant errors, thereby greatly influencing the cost and integrity of drainage structures.

This section discusses methods for determination of watershed characteristics affecting the assessment of runoff.

Watershed Area

The area of a watershed is a fundamental variable in all flow calculation methods. If an area does not contribute to runoff, such as lands draining into depressions that have no surface outlets, it should be discounted from the total. The net area is referred to as the effective area. Based on prevailing hydrologic conditions, the effective area can change with various storms, depending on the storm frequency and magnitude, and on the potential for ponding in depressions. For small magnitude storms certain areas may not contribute to flow.

Airphoto interpretation should be used for delineating basins, wherever possible. It is essential to investigate significant uncertainties in watershed boundaries and suspected flow diversions. Where mosaics are not available, provincial topographical and municipal drainage maps may be used. The scale of the map should be related to the level of accuracy of area measurements required.

Watershed areas should be adjusted to account for diverted flows between adjacent areas. Diversions may include:

- existing natural diversions; and
- potential overflow sites.

In some urban areas, major and minor systems may discharge to different outlets, and as such, diversions may vary with the intensity of the storm.

Watershed Length

The length of a watershed has an effect on the time of concentration, t_c, and therefore, the peak flow. The length may be defined as the longest drainage path within the watershed. It includes the defined and undefined flow path within the watershed. This may not necessarily coincide with the drainage path along the main drainage channel. A practical approach in deciding the critical path is to check for all possible critical paths along several possible drainage paths to determine the longest one.

Longer watershed lengths with the same drainage area give peak flows of less magnitude and longer time to peak.

Runoff Coefficient

The combination of soil types and land use has an important influence on the magnitude of floods. This effect is quantified in runoff calculations in terms of the runoff coefficient.

The runoff coefficient (C) is the ratio of the depth of runoff to the corresponding depth of rainfall falling on an area. It indicates the runoff potential of a particular combination of soil, land use and topography. The value of C ranges from less than 0.1 for sandy woodland to 0.95 for paved areas. Design Chart 1.07 gives the runoff coefficient for different types of land uses.

An arithmetic weighted C is used where there are different land uses and soil types to obtain a composite runoff coefficient, as follows:

$$C = \frac{A_1C_1 + A_2C_2 + \cdots\cdots}{A_t} \tag{8.10}$$

where:

C = composite runoff coefficient
$A_{1,2,...}$ = area corresponding to specific land use or soil type, ha
$C_{1,2,...}$ = runoff coefficient corresponding to $A_{1,2,....}$
A_t = total drainage area, ha

For urban areas, the values of the runoff coefficient may be increased for the high magnitude storms under urban conditions. For the 25, 50 and 100-year events, it is recommended to increase the coefficient by 10, 20 and 25% respectively (MTO 1986). No adjustments are recommended for rural drainage areas.

Example 8.2: Calculation of Runoff Coefficient

Required

Calculate the composite runoff coefficient for a 12 hectare watershed with the characteristics given below. Adjust the runoff coefficient for a 25-year event:

Given

The watershed is subdivided into 4 subareas based on land use.

Area, A	Land Use	Topography/Soil	Runoff Coeff. (Design Chart 1.06)
5 ha	heavy industrial	flat/clay	0.90
3 ha	multiple attach. res.	flat/clay	0.70
4 ha	single family res.	rolling/sand loam	0.35
2 ha	woodland	rolling/loam	0.30

Solution

Substitute values of A & C into Equation 8.10:

$$C = \frac{A_1C_1 + A_2C_2 + A_3C_3}{A_t} \qquad (8.10)$$

$$= \frac{5 * 0.90 + 3 * 0.70 + 4 * 0.35 + 2 * 0.30}{5+3+4+2} = 0.61$$

For a 25-year event, increase the runoff coefficient by 10% (Design Chart 1.07)

$C = 1.10 * 0.61 = 0.67$

Soil Types and Land Use

The combined effect of soil type and texture, and land use has an important influence on the magnitude of floods. This effect can be quantified by the Curve Number (CN) system developed by the U.S. Soil Conservation Service (1972).

Curve Number

The Curve Number system of measuring runoff volume was developed for simulating rainfall runoff in agricultural lands, and does not take into consideration frozen ground. The basic idea is to assign a CN to indicate the potential abstractions of the drainage area given the soil type, land use and the antecedent moisture condition, AMC I, II or III, which corresponds to dry, average and saturated soils respectively.

The CN can be calculated using:

$$CN = 25400 / (254 + S) \qquad (8.11)$$

where:

S = storage within watershed, depth, mm

However, since the storage within a watershed in not often known and difficult to quantify accurately, a more practical and consistent approach is to use the soil type and land use information to determine a CN value. This is determined in the following way:

- determine the soil texture from soil maps or airphoto interpretation;
- classify the soil according to the Hydrologic Soil Group using; (Design Chart 1.08)
- determine the land use, possibly from airphotos; and
- determine the CN corresponding to the soil type and land use (Design Chart 1.09)

Soil Type and Texture

Soil types and texture may be determined from soil maps, geotechnical investigations or airphoto interpretation.

When delineating the various soil types and textures on maps, areas of soil covered by wetland should be excluded, as they should be counted as wetland.

Hydrologic Soil Group

The hydrologic soil group is used to classify soils into groups of various runoff potential.

The Soil Conservation Service (SCS) classifies bare thoroughly wet soils into four hydrologic soil groups (A, B, C and D). SCS descriptions of the four groups, modified slightly to suit Ontario conditions, are as follows: (Design Chart 1.09)

A: High infiltration and transmission rates when thoroughly wet, eg. deep, well drained to excessively-drained sands and gravels. These soils have a low runoff potential.

B: Moderate infiltration and transmission rates when thoroughly wet, such as moderately deep to deep open textured loam.

C: Slow infiltration and transmission rates when thoroughly wet, eg. fine to moderately fine-textured soils such as silty clay loam.

D: Very slow infiltration and transmission rates when thoroughly wet, eg. clay loams with a high swelling potential. These soils have the highest runoff potential.

In Ontario, soils have been found to lie between the main groups given above, and have therefore been interpolated as AB, BC, CD as appropriate, such as Guelph loam, which is classified as BC.

In determining the hydrologic soil group, it is advisable in cases of doubt to use the higher classification rather than the lower to avoid underestimating the design flood.

Land Uses

Hydrologically similar land uses have been grouped as follows:

Crop	-	all areas under cultivation, including summer fallow; includes also, small areas of improved land such as farm land and yards;
Pasture	-	seeded and natural pasture, and other unimproved farmland;
Woodland	-	farm woodlot, tree farm, land leased for cutting, bush and cut-over land with young trees. Large tracts operated as a separate business from the farm. Variations in the type of woodland should be ignored.

Soil Moisture

One of the factors that influence runoff on a pervious surface is the amount of moisture within the soil. The amount of rainfall that infiltrates into the soil is controlled by the soil moisture storage capacity and the state of wetness of the soil.

Antecedent Moisture Condition (AMC)

Due to the lack of records on soil moisture, the antecedent moisture condition is measured as an index of the Antecedent Precipitation Index (API). The measurement is based on the premise that soil moisture is depleted at a rate proportional to the amount of storage in the soils.

Kohler and Linsley (USBR Paper 34, 1951) proposed the following equation to describe API for recorded storms:

$$API_i = K * API_{i-1} + P_i \tag{8.12}$$

where:

API_i = API for day i
P_i = precipitation recorded on day i
K = recession constant, varies between 0.85 to 0.98

Curve Number Adjustment Approach

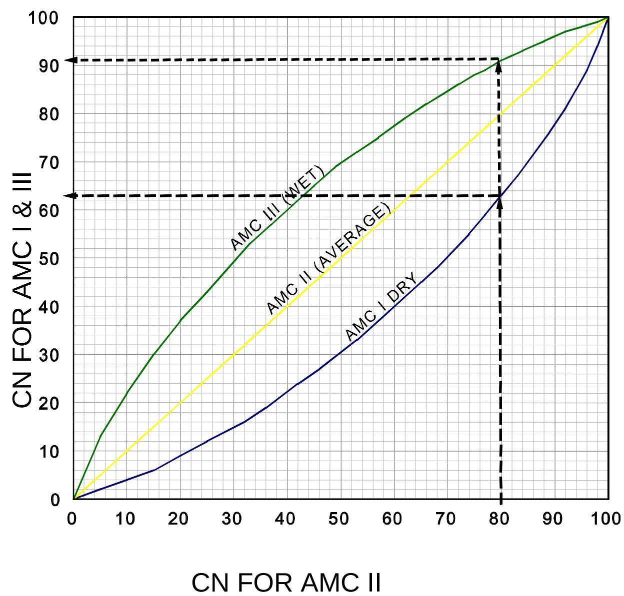
The CN for a given soil varies with the initial antecedent moisture conditions (AMC). The three initial soil moisture conditions are:

- AMC I - dry
- AMC II - average moisture
- AMC III - saturated.

CN may be converted from one AMC to another using Figure 8.5, or Design Chart 1.10.

As was discussed earlier in this chapter, when applying the 48-hour storm duration for Hurricane Hazel (Zone 1) and the 12-hour Timmins Storm (Zone 3), the soil moisture condition is specified as being AMC II. However, if the 12-hour Hurricane Hazel is used (last 12 hours of the storm), the CN must be converted to AMC III to account for the rainfall in the preceding five days (saturated condition).

Figure 8.5: Variation of CN with Antecedent Moisture Condition



(MTO, 1986)

Lakes and Wetlands

In determining the overall CN for a watershed, it is necessary to assign a value to lakes and wetlands. A CN of 50 is proposed for this purpose.

Example 8.3: Calculation of Curve Number and Effect of Soil Moisture

Required

Determine the representative SCS curve number (CN) for a homogeneous, wooded watershed adjacent to Lake Erie near Port Colborne for analysis of a 12-hour regional storm event.

Given

- The soil in this area is predominantly clay, has a permanently high water table and only a thin layer of soil over bedrock.
- The land use is woodland with good coverage.
- The watershed is in Zone 1. (Design Chart 1.02)

Solution

- Determine the hydrological soil group. Since the soil in this area is predominantly clay, the hydrological group D is appropriate. (Design Chart 1.08)
- The watershed is in Zone 1, therefore, Hurricane Hazel applies. When using the 12-hour storm distribution the CN must be converted to AMC III. However, AMC II is appropriate if the entire 48-hour storm distribution is applied.
- The CN for woodland, soil group D is 77 for AMC II (Design Chart 1.09)
- CN = 77 (for AMC II) converts to CN = 89 (for AMC III) (Design Chart 1.10)

Therefore, CN = 89 will be used in the analysis.

Slope of Watershed

The slope of a watershed has a considerable effect on the time of concentration, t_c, and therefore, on the peak runoff.

For the purpose of calculating runoff, the watershed slope is taken as the representative slope along the longest flow path, previously defined as the length of the watershed.

Two methods are presented for determining watershed slopes: the 85/10 method; and the equivalent slope method.

The 85/10 Method

This method is generally recommended for normal use. It usually provides a representative slope of a stream channel upstream from a site of interest. It avoids the distorting effects of a steep upper portion of a watershed or the effects of a highly irregular or convex or concave profile. From the site of interest on a stream channel, points are located on the main channel, 10% and 85% of the main channel length upstream to the edge of the watershed. The elevation difference between these two points and the total length of the main channel from the site to the head of the watershed are used to determine the average slope using Equation 8.13. The heights and lengths of falls and rapids are subtracted from both the height and length difference in this calculation, such that the characteristics of the river are more accurately represented.

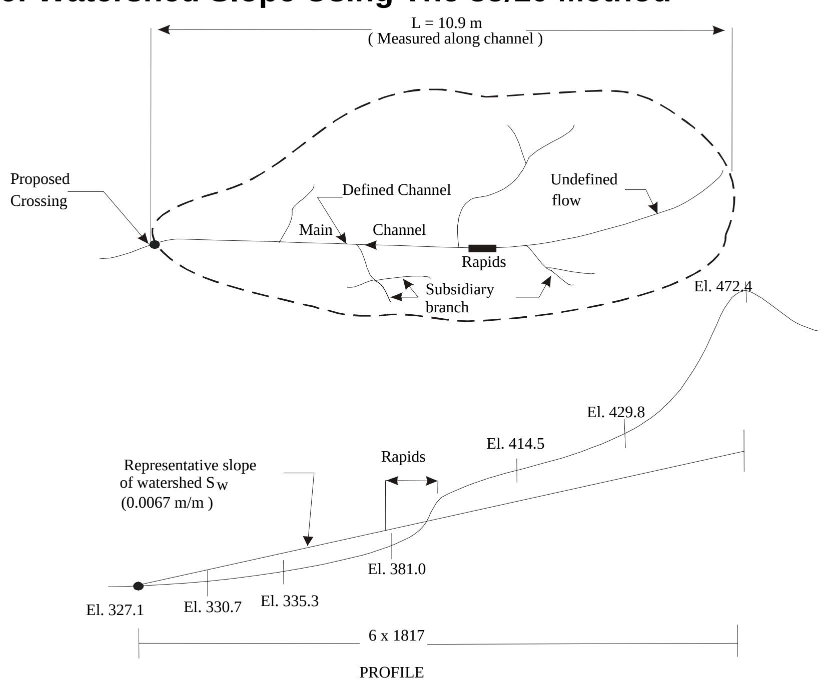
The 85/10 formula is:

$$S_w = \frac{100 * (\Delta h - h_f)}{0.75L - L_f} \qquad (8.13)$$

where:

- S_w = watershed slope, %
- Δh = difference in elevation, m, between the 85% point and the 10% point obtained from contours, airphotos, etc.
- h_f = sum of heights of rapids and waterfalls between 10% and 85% points, m
- L = total length of main channel, includes the undefined flow path, to head of basin, m
- L_f = sum of lengths of rapids and waterfalls, up to 10% of L, m

Figure 8.6: Watershed Slope Using The 85/10 Method



(MTO, 1986)

The Equivalent Slope Method

In this method, the watershed slope is defined as the uniform slope which would produce the same travel time of the flow as that created by the actual main channel profile. It gives relatively accurate results for streams with highly irregular, strongly convex, or concave profiles or with extensive falls and rapids.

With this method, the stream profile is divided into reaches, of approximately equal lengths, to reflect major irregularities in the profile. The required slope is then computed as the weighted average of the slopes of the individual reaches.

The formula is as follows:

$$S_w = 100 \left[n / \Sigma (S_n^{-0.5}) \right]^2 \qquad (8.14)$$

where:

S_w = watershed slope, %
S_n = slope of an individual reach of the channel, m/m
n = number of reaches of approximately equal length

Example 8.4: Calculation of Slope of Watershed

Required

Determine the slope of the watershed in Figure 8.6 by the 85/10 and equivalent slope method:

Solutions

85/10 Method:

Substitute values of h, h_f, L, L_f in Equation 8.13:

$$S_w = \frac{100 * (\Delta h - h_f)\%}{0.75L - L_f} \qquad (8.13)$$

$$= \frac{100 * (100.9 - 30.5)\%}{(0.75 * 10900) - 400}$$

$$= 0.91\%$$

Therefore, the watershed slope 0.91%

Equivalent Slope Method:

1. Divide the channel into approximately equal lengths as shown in figure.
2. Calculate the inverse square root of the slope for each section. ($S_n^{-0.5}$)

Elevation (m)	Fall (m)	Length (m/m)	Slope, S_n	$S_n^{-0.5}$
327.1	15.9	2725	0.0058	13.13
343.0	23.0	2725	0.0084	10.91
366.0	39.0	2725	0.014	8.45
405.0	67.4	2725	0.025	6.36
472.4				
Σ	145.3	10900		38.85

Substitute n and $(\Sigma S_n)^{-0.5}$ into Equation 8.14 to determine the equivalent slope

$$S_w = 100\,[n / \Sigma(S_n^{-0.5})]^2\ \% \tag{8.14}$$
$$= 100\,[4 / 38.85]^2\ \%$$
$$= 1.06\ \%$$

Therefore, the equivalent slope is 1.06 %

Watershed Time of Concentration

The time of concentration, t_c, is a controlling factor on peak flow estimation. It is defined as the time it takes a wave to travel from the hydraulically "farthest" point of a watershed to the location downstream where the flow rate is to be calculated. Theoretically, at this time, the entire watershed upstream from the point of interest would be contributing runoff. The hydraulically "farthest" point is that with the largest travel time and not necessarily the greatest distance.

If the design storm duration is less than t_c, the runoff will be less than maximum since not all of the watershed area will be contributing.

The time of concentration, $t_{c,}$ of overland flow component (sheet flow) and for channel flow component is additive. In storm sewer design, the overland component is referred to as the inlet time.

Several methods are available to determine t_c. Two recommended approaches are:

- the Bransby Williams formula, and
- the Airport formula.

Bransby Williams Formula

In watersheds with a runoff coefficient, C, greater than 0.40, the Bransby Williams formula is one of the more accepted methods. The method considers area, length and slope of a watershed as follows:

$$t_c = 0.057 * L / (S_w^{0.2} * A^{0.1}) \qquad (8.15)$$

where:

t_c = time of concentration, min
L = watershed length, m
S_w = watershed slope, %
A = watershed area, ha

Design Chart 1.11 gives solutions to the Bransby Williams formula directly, and provides an example of its application.

Airport Formula

For watersheds where the runoff coefficient, C, is less than 0.40, the Airport formula gives a better estimate of t_c. This method was developed for airfields and is expressed as follows:

$$t_c = \frac{3.26 * (1.1 - C) * L^{0.5}}{S_w^{0.33}} \qquad (8.16)$$

where:

t_c = time of concentration, min
C = runoff coefficient
S_w = watershed slope, %
L = watershed length, m

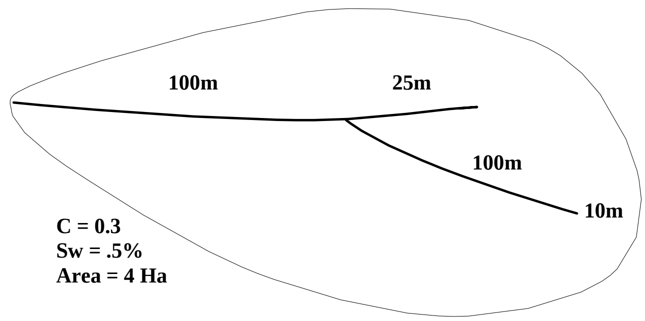
When a watershed length is made up of widely differing surfaces (e.g. grass and concrete), t_c, can be calculated for each surface, and the individual values summed to give the overall value.

Design Chart 1.12 may be used to solve for t_c, using the Airport Formula for watershed lengths less than 350 m.

Example 8.5: Calculation of Time of Concentration

Estimate the time of concentration, t_c, for the watershed illustrated below, for two runoff coefficient values, 0.3 and 0.6.

Given



Solution

Airport Formula; recommended when C = 0.3:

$$t_c = \frac{3.26 * (1.1 - C) * L^{0.5}}{S_w^{0.33}} \tag{8.16}$$

$$t_c = \frac{3.26 * (1.1 - 0.30) * 210^{0.5}}{0.5^{0.33}}$$

$$= 47.5 \text{ min}$$

Bransby Williams Formula; recommended when C = 0.6:

$$t_c = 0.057 * L / (S_w^{0.2} * A^{0.1}) \tag{8.15}$$

$$t_c = 0.057 * 210 / (0.5^{0.2} * 4^{0.1})$$

$$= 12.0 \text{ min}$$

Watershed Abstractions

Abstractions refer to that portion of precipitation that gets stored in the watershed and does not contribute to the runoff process. Abstraction losses include interception, infiltration to soil moisture storage, and depression storage.

The total rainfall less losses or abstractions is referred to as the effective rainfall. It is used in hydrologic models to calculate the design flow rates. The estimation of hydrologic abstractions is, therefore, very important in the final determination of the design flow. If abstractions are underestimated, the design flows will be overestimated, and drainage systems may be oversized.

Abstractions are dependent on the topography of the drainage area, soil cover and initial moisture conditions.

Infiltration

Infiltration is the movement of water from the ground surface into the soil. The rate and quantity of infiltration depend on the permeability and voids respectively, within the soil structure under dry conditions. For wet conditions, the degree of saturation is a major factor regulating the amount that can be abstracted.

The most widely used methods for calculating infiltration losses include the Horton's infiltration equation, the U.S. SCS Curve Number and the Proportional methods. Table 8.3 summarizes the three methods.

Design Chart 1.13 gives typical infiltration values for the Horton method.

Watershed Storage

The input-output (rainfall-runoff) process within a watershed is greatly affected by the watershed storage potential.

Storage has the effect of decreasing and delaying peak flows and extending the duration of the hydrograph by temporarily retaining water in storage.

Watershed storage may include:
- ponds, swamps, depressions and lakes;
- channel and floodway;
- snow pack; and
- subsurface storage.

The storage within basin depressions should be carefully assessed. For low magnitude and frequent events, depressions can considerably reduce the effective flow area. Nevertheless, for high magnitude events they may be ineffective in abstracting any runoff as they may be already full.

The impact of depressions is more evident in watersheds where poorly developed watercourses exist; and flow cascades from one depression to another before it reaches the stream. Depressed areas will contribute runoff to the stream only if the runoff volume is greater than their storage capacities.

Depression storage contributes to baseflow, hence low flow to the stream depends on the potential storage within the depressions and the permeability of the subsurface soils. This makes these watershed features an important ecological consideration.

One way of accounting for basin storage is to apply a basin storage factor to the unadjusted peak discharge as follows:

$$Q = K_s * Q_u \qquad (8.17)$$

where:

Q = adjusted peak flow, m^3/s
K_s = reduction factor (Design Chart 1.06)
Q_u = unadjusted peak flow, m^3/s

This method can be used with the Rational Method. Design Chart 1.06 provides guidelines for estimating depression storage. The values should be used cautiously if estimates of the hydrologic peak flow and runoff are sensitive to depression storage.

A generally accepted way of representing the total abstraction due to storage is to express the ratio of runoff volumes as an index. One such index is the volumetric runoff coefficient. A coefficient equal to 1 indicates that there is no basin storage.

Watershed storage is accounted for in different ways depending on the method of calculation the runoff flow rate. Therefore, further discussion of other methods for calculating watershed storage will be covered in this chapter in the discussions of the different runoff calculation methods.

Table 8.3: Infiltration Procedures

METHOD	EQUATION	PARAMETERS	REMARKS
U.S. SCS CURVE NUMBER METHOD	$Q = \frac{(P - Ia)^2}{(P + S - Ia)}$	Q = runoff depth, mm P = precipitation, mm S = soil storage capacity, mm $= \frac{25400}{CN} - 254$, mm CN = curve number based on vegetative cover and four soil groups (A, B, C and D) Ia = initial abstraction, mm	Estimates depth of runoff from rural and natural basins; has been widely applied to management of watersheds and floodplains and design of hydraulic control structures; initial abstraction of 0.2*S has been questioned for Ontario conditions; can be varied in OTTHYMO and MIDUSS.
HORTON INFILTRATION METHOD	$f_t = f_\infty + (f_o - f_\infty)e^{-kt}$ if $i < f_t$ then $f = i$	f_t = infiltration rate, mm/hr f_∞ = minimum infiltration rate, mm/hr f_o = maximum infiltration rate, mm/hr e = natural logarithm k = decay coefficient, 1/s t = time from beginning of precipitation, s i = rainfall intensity	Estimates precipitation losses due to infiltration from urban areas; widely used; f_∞ is a function of soil properties, initial infiltration is a function of antecedent precipitation; f_∞ and f_o vary for different soil types.
GREEN-AMPT INFILTRATION METHOD	For $F < F_s$ $f = i$ for $F > F_s$ $f = f_p = K_s\left(1 + \frac{S_u * IMD}{F}\right)$	F = cumulative infiltration volume, mm F_s = cumulative infiltration volume required to cause surface saturation, mm $= \frac{S_u * IMD}{i / K_s - 1}$ for $i > K_s$ = no calculation for $i < K_s$ f = infiltration rate, mm/s f_p = infiltration capacity, mm/s i = rainfall intensity, mm/s K_s = saturated hydraulic conductivity, mm/s S_u = average capillary suction at the wetting front, mm IMD = initial moisture deficit for this event, mm/mm	Second (last) step estimates infiltration based on the Estimates infiltration rate; first step predicts volume of water that infiltrates before saturation; Green Ampt Equation; infiltration is a function of surface moisture conditioning; input parameters K_s, S_u, and IMD vary for different soils.

(MTO, 1989)

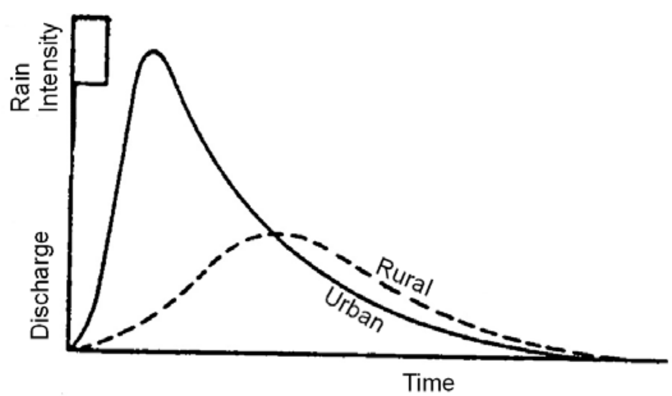
Differences in Urban and Rural Watersheds

Urban type basins yield floods that are characteristically different from those generated from rural basins. The differences are mainly a reduction in the time to peak, an increase in peak flow and runoff volumes for the urban type basin. Figure 8.7 illustrates the fundamental differences in the rural and the urban hydrographs.

The differences are due to the fact that, before urbanization, a large portion of the total rainfall infiltrates into the soil, and is stored in depressions. After urbanization, those abstractions are minimized and the volume and rates of runoff are increased due to higher imperviousness and a more efficient conveyance system. Characteristically, the urban hydrograph is narrower, and peaky. The shape of the urban hydrograph may sometimes be altered, and becomes similar to the rural hydrograph, if there is detention storage within the urban watershed. For this reason, storage facilities are used as means of controlling post-development flows.

It should be recognized that controlling post-development flow rates with detention storage can actually increase downstream peak flow rates. This could occur due to the timing of arrival of the peak flows at a point downstream from various portions of the watershed. If the peak flow from one portion of the watershed arrives at the same time as the peak flow rate from another portion of the watershed the resulting peak flow may be greater than if, with no control, higher peak flows arrive at the same point sequentially from the same two portions of the watershed.

Figure 8.7: Urban and Rural Hydrographs



Estimation of Design Floods

There are several reasons why a flood flow rate may be required in the highway planning and design process, including sizing of conveyance systems, calculating water levels, assessing the impacts of urbanization, delineation of a floodway, design of channel improvements, removal or inclusion of storage within a watershed, etc.

This section provides the designer with an overview of various types of design floods and explains the procedures for estimating them. Further, this section demonstrates how the rainfall runoff process is transformed into a discharge or design flow output. A number of computational methods are discussed to provide designers with an awareness of these applications. Both simulation and statistical methods of calculating flows are discussed.

The Design Flood

A design flood for a highway facility is based on some measure of acceptable risk in consideration of liabilities associated with failure and, most importantly, the safety of the public. Drainage works for highway projects, like all other drainage infrastructures, are designed for a real or hypothetical storm event that may or may not happen during the lifetime of the structure. Storms larger than that design storm event may result in failure or damage. To establish an absolute zero probability of failure or damage during a project lifetime may be cost-prohibitive and impractical.

Design criteria will vary with the type of project, in recognition of the impacts of failure. For example, a spillway will have a low probability of failure, while a ditch will have a high probability of being overtopped. In establishing a design flood, factors such as cost, the importance of the facility, the hydraulic integrity of the structure, and the human consequences of failure are factored in establishing the risk, and hence the design flood magnitude.

Various design storms have been established in keeping with the *Provincial Flood Plain Guidelines* (MNR, 1988), and the relative importance of various drainage system components (ditches, sewers, culverts, etc.) and the level of protection based on the need for maintaining highway use during major storms. Major highways would require a higher level of protection, and hence, design storms characteristic of an infrequent occurrence would apply.

Return Period and Probability of Occurrence

In flood estimation, the term return period, T_r, also referred to as the frequency, is the average number of years between occurrences of a discharge equal to or greater than a given rate.

The probability of occurrence is the reciprocal of the return period. For example, a 25-year flood, or a flood with a return period or frequency of 25 years, has a probability of occurrence, in any given year, of 1/25 or 4%.

Determining Risk in Drainage Design

At every river and stream crossing, a decision has to be made regarding the return period of the design flood to be adopted for design. Implicit in this decision is acceptance of a calculated risk of flood exceedance during the life of a project. The consequences of assuming such risk and associated failure or damage should be determined based on adopting acceptable engineering practice.

Risk in drainage is closely linked to a specified return period flood during the design life of a project. For example, a 5% risk is often used in design of temporary river diversions and cofferdams over their life times.

Risk is usually expressed as a probability, P, that a flood will be exceeded in any one-year period and can be expressed as:

$$P = 1 - (1 - 1/T_r)^n \tag{8.18}$$

where:

T_r = return period of the storm, years
n = life of the structure, years

Risk factors for various flood frequencies in relation to the life of the structure are given in Table 8.4. As an example, from the table it can be seen that, during the 50-year life of a culvert, there is a 99% risk that a 10-year flood will be equalled or exceeded and there is a 40% risk that a 100-year flood will be equalled or exceeded.

Joint Probability

Certain extreme flows or water levels may result from the simultaneous occurrence of more than one event.

For example, flooding of a certain river may depend on a combination of releases from an upstream dam, extreme rainfall, snowmelt or high water levels in a lake. The occurrences of these independent events may have to be expressed in a joint probability to assess the risk associated with flooding problems. The determination of a joint probability requires the knowledge of local climatic and geographic conditions of the site.

Table 8.4: Risk Factors

Average Return Period (yrs)	Probability of Exceedance During: n-years of life of structure (percent)					
	2.3	5	10	25	50	100
2.33	73	94	100	100	100	100
5	41	67	89	100	100	100
10	22	41	64	93	99	100
25	9	18	34	64	87	98
50	5	9	18	41	64	87
100	2	5	9	22	40	64
1000	0	1	1	3	5	10

Example 8.6: Assessment of Design Risk

Required

Determine the return period for the design of a cofferdam for a temporary river diversion that is proposed to be in service for 2 years.

Given

A 2% risk of exceeding the design flood has been considered acceptable within the life of the structure .

Solution

$$P = 1- (1 - 1/T)^n \qquad (8.18)$$

where:

$n = 2$

$P = 0.02$

Substituting values for P and n:

$0.02 = 1 - (1 - 1/T)^2$
$0.98 = (1 - 1/T)^2$
$0.01005 = 1/T$
$T = 99.5$ years (~ 100-year event)

Therefore, the 100-year design event is appropriate.

Design Flood Estimation Methods

The rainfall-runoff transformation methods calculate runoff from a design rainfall, taking into account hydrologic losses such as initial abstraction and infiltration and the frequency of occurrence of floods. Generally, rainfall data are more easily obtained than runoff measurements in streams, and therefore such methods that use design rainfall are frequently used. Nevertheless, results of the rainfall-runoff approach are considered less accurate than those involving methods that utilize streamflow records.

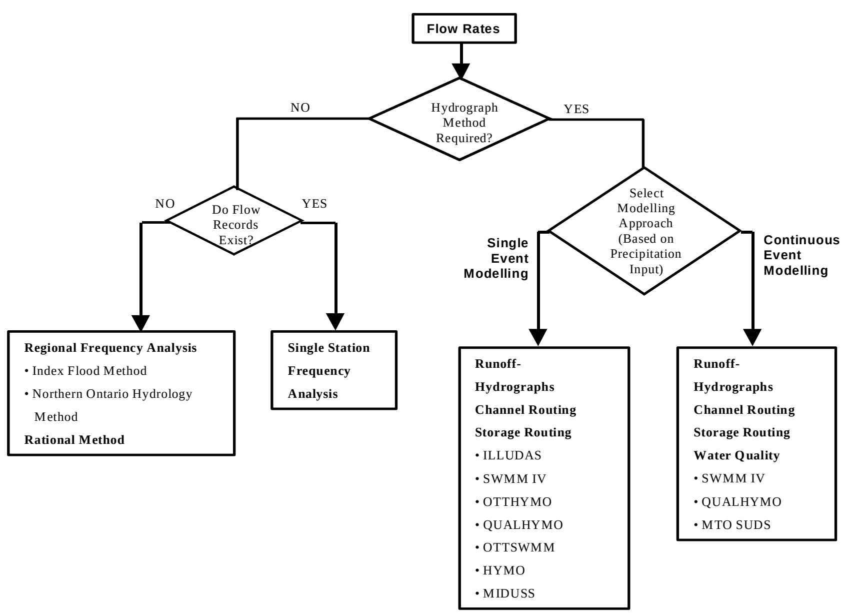
Several methods are available which can be grouped into broad categories:

- rainfall-runoff transformation methods; and
- analysis of streamflow records.

Figure 8.8 provides a listing of the different methods under each of the above two groups. The following sections will present the theoretical principles of the following methods:

- The Rational Method
- Regional Frequency Analysis Methods
  - Modified Index Flood Method
  - Northern Ontario Hydrology Method
- Single Station Frequency Analysis Method
- Hydrograph Methods

Figure 8.8: Methods For Estimating Design Flow Rates



The discussion on the Hydrograph Methods provides the theoretical principles generally utilized by these methods with no reference to particular computer models. For specific details on the different computer models, refer to the model summary sheets in Appendix A of this chapter and to the user's manual for each of the models.

The Rational Method

The Rational method is one of the earlier developed methods of calculating peak flows. In spite of the availability of advanced computational techniques, it remains a valid approach to peak flow estimation for small drainage areas. The application of this method should be limited to watersheds less than 100 hectares in size.

Some applications of the Rational Method include:

- determination of peak flows to size channels, sewers, ditches and culverts;
- preliminary design estimation for drainage systems;
- flow estimation to design erosion and sediment control devices.

The method is expressed as follows:

$$Q = 0.0028 * C * i * A \qquad (8.19)$$

where:

Q = peak runoff rate, m^3/s
C = weighted runoff coefficient for the catchment area (Design Chart 1.07)
i = rainfall intensity, mm/h
A = drainage area, ha

Assumptions inherent in the Rational method are:

- the peak rate of runoff, Q, is determined by using an average rainfall intensity, i, over the entire watershed with a time duration equal to the watershed time of concentration, t_c.
- the peak rate of runoff is assumed to have a return period equal to that of the intensity-duration-frequency curve;
- the rainfall intensity, i, remains constant for the computed time of concentration, t_c, and is uniform across the drainage area;
- the runoff coefficient, C, does not vary over the duration of the storm.

A computer model of the Rational Method has been developed by MTO, *MTO Rational Drainage Model*, to assist in the application of the method.

Example 8.7: Rational Method

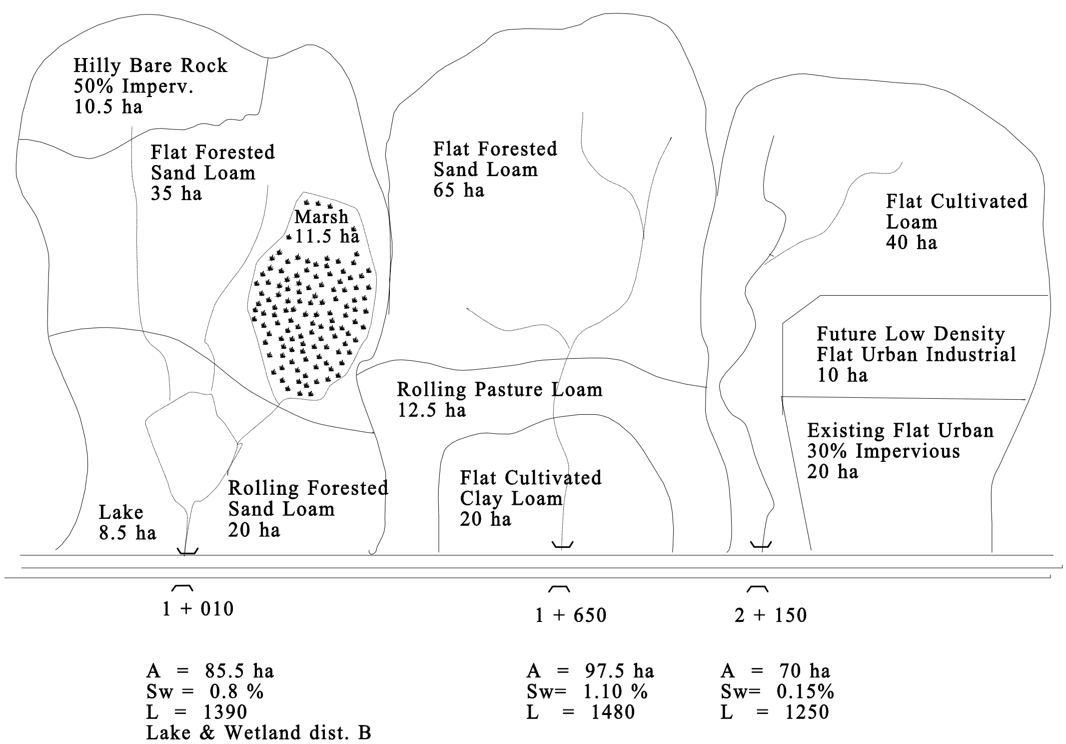
Required

Determine the design peak flow for three undersized culverts that are to be replaced as part of the reconstruction of a section of highway north of Toronto.

Given

The watersheds and relevant data are shown in the figure below. The nearest climatological station is Person International Airport. The Intensity-Duration-Frequency (IDF) curve for this station is shown in Figure 8.3

Plan of Watersheds



Solution

1. Determine the Composite Runoff Coefficients

For each culvert determine the runoff coefficients C, for each sub-watershed (Design Chart 1.07). Substitute into Equation 8.19 to obtain the composite runoff coefficient at each culvert.

1 + 010 $C = \frac{(38 \times 0.08) + (20 \times 0.12) + (10.5 \times 0.70) + (20 \times 0.05)}{85.5}$

$= 0.16$

2 + 650 $C = \frac{(65 \times 0.08) + (12.5 \times 0.35) + (20 \times 0.55)}{97.5}$

$= 0.21$

3 + 150 $C = \frac{(20 \times 0.40) + (10 \times 0.60) + (40 \times 0.35)}{1400}$

$= 0.40$

2. Determine the Time of Concentration, t_c

For station 1+010 and 2+650 the Airport method is used to calculate the time of concentration since C < 0.4. For station 3+150 the Bransby Williams method is used.

$t_{c\,(1+010)} = 122.9$ min
$t_{c\,(1+650)} = 108.1$ min
$t_{c\,(2+150)} = 68.0$ min

3. Determine Rainfall Intensity, i

From Figure 8.3, the rainfall intensity is that corresponding to an event duration equal to the time of concentration, t_c, of the watershed and the 25-year return event:

$i_{(1+010)} = 25.0$ mm/h
$i_{(1+650)} = 27.4$ mm/h
$i_{(2+150)} = 37.9$ mm/h

4. Calculate the Design Peak Flow

Substituting into the Rational Method equation to calculate the design peak flows:

$$Q = 0.0028 * C * i * A \qquad (8.19)$$

$Q_{(1+010)} = 0.0028 * .16 * 25.0 * 85.5$
$= 0.96\ m^3/s$

$Q_{(2+324)} = 0.0028 * .21 * 27.4 * 97.5$
$= 15.6\ m^3/s$

$Q_{(3+428)} = 0.0028 * .40 * 37.9 * 70$
$= 2.97\ m^3/s$

The watershed of station 1+010 includes storage due to lakes and wetlands; therefore the peak flow must be adjusted to allow for the attenuation of peak flows as follows:

Percentage (%) of lake/wetland = 20ha / 85.5ha * 100%
= 23.4%

For type B (watershed); $K_s = 0.56$ (Design Chart 1.06)

Therefore, $Q_{(1+010)} = 0.56 \text{ x } .96\ m^3/s = 4.7\ m^3/s$

Regional Frequency Analysis

These methods utilize regional watershed and climatic characteristics to calculate peak flows. They are easy to apply, require limited data, and are widely used for ungauged watersheds. They are some of the most accurate methods available for analysis of medium to large rural watersheds with design flow return periods up to 100 years. The most common of these methods are:

- The Modified Index Flood Method, and
- The Northern Ontario Hydrology Method.

The Modified Index Flood Method

The Index Flood Method was developed by the U.S. Geological Service (Dalrymple, 1960) and modified for MTO in 1986 to reflect Ontario conditions. This method has since been referred to as the Modified Index Flood Method. In 1992 the MTO commissioned the School of Engineering, University of Guelph to conduct a review of the Modified Index method to improve the accuracy of the method. Details of results of the study and suggestions for improvements in its application are provided in the MTO publication, *Report on the Evaluation and Suggestions for Improvements to the Modified Index Flood Method* (Joy and Whiteley, 1996). This update is adopted in this manual and supersedes the method described in Chapter H of the previous edition of the Drainage Manual.

The development of the index flood method involved developing a regression equation expressing an index flood, of a specific return period, in terms of watershed characteristics (slope, shape, detention) and climate, within a homogeneous region. The modification of the Index Flood Method (MTO, 1986), modifies the USGS method of using the 2-year index flood, and uses instead the 25-year flood, the peak flood with an annual probability of being equaled or exceeded of 4%. The modification is applied because the 25-year event is the most widely used return period for bridges and culverts. Also, the 25-year Index Flood curves represent the average basin characteristics. For floods of other return periods (T), a ratio of Q_T/Q_{25} can be used to estimate Q_T.

MTO (1986) developed the 25-year index flood from flow records in the province for the 25-year storm for medium-sized and large watersheds based on a regional frequency analysis of annual floods recorded at Ontario stream gauging stations. In 1996, the accuracy of the method was evaluated by comparing the flow rate estimates from the MIFM to flows obtained from flood-frequency analysis of 49 gauged watersheds representative of the stream systems in Ontario.

The general equation of the 25-year Index Flood curve is determined as:

$$Q_{25} = C_{25} * A^{0.75}, \text{ m}^3/\text{s} \qquad (8.20)$$

where:

C_{25} = watershed class coefficient
A = basin area, km^2

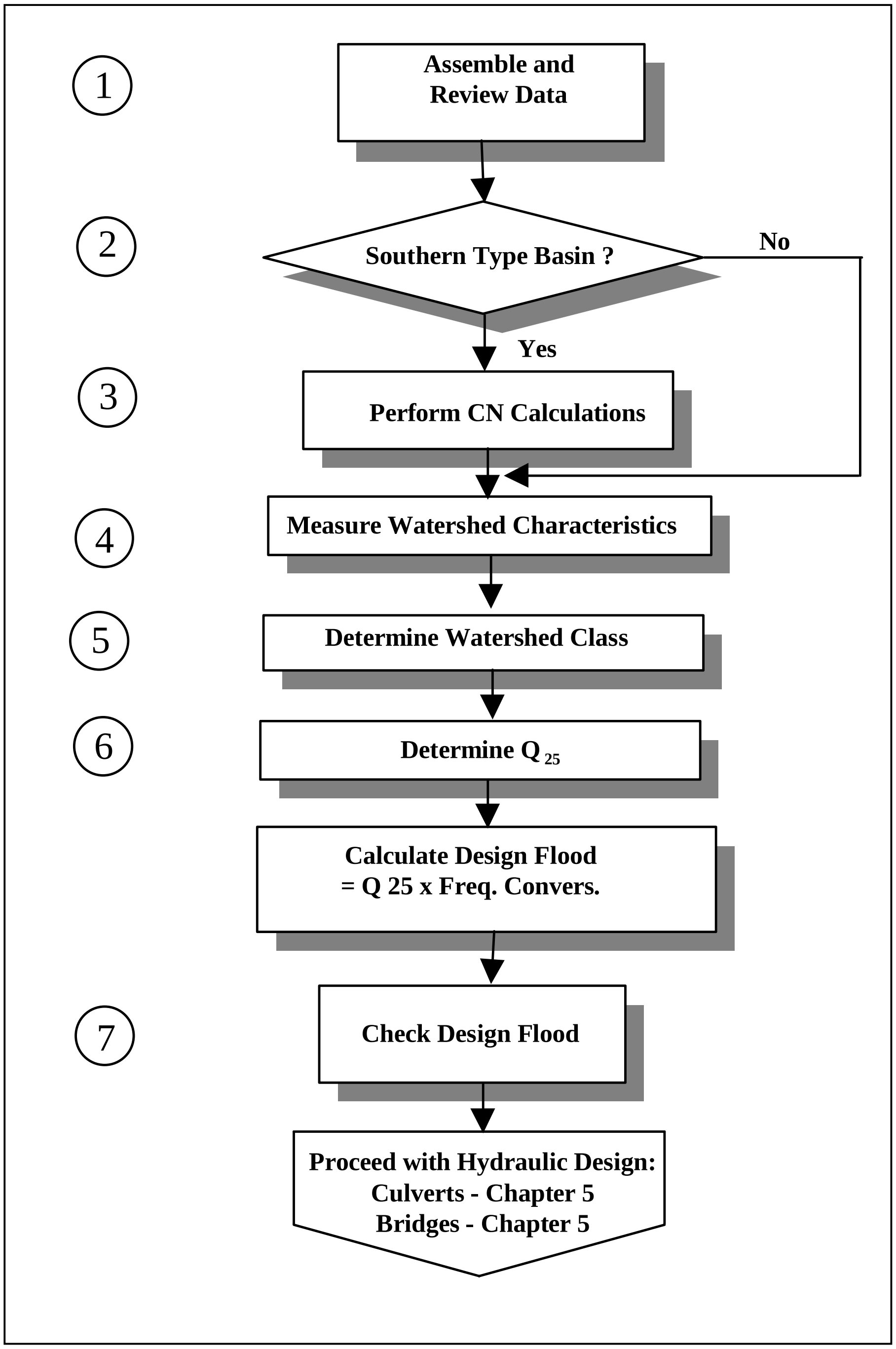
The procedure for estimating the flow using the MIFM can be summarized in the following 4 steps.

- Determine whether the watershed is of the Canadian shield or southern Ontario type, or both
- Measure the appropriate watershed characteristics.
- Determine the watershed class, as shown in the example.
- Determine the 25-year (index) flood and adjust it to the required frequency.

A detailed application of this method is presented in the example below.

It is recommended that the designated application for the MIFM be for watersheds larger than 25 km^2. For watersheds between 5 km^2 and 25 km^2, results from the MIFM may be applicable but should be compared to those obtained from techniques developed explicitly for smaller watersheds.

Figure 8.9: Flow Chart of Modified Index Flood Method

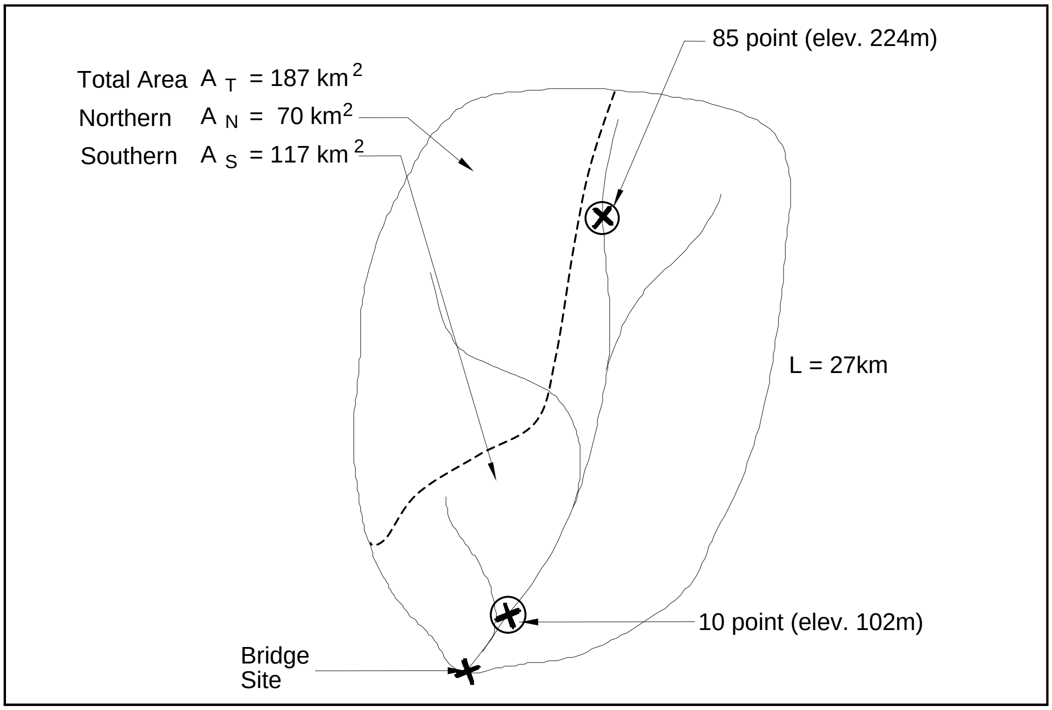


(MTO, 1986)

Example 8.8: Estimating Flows Using the Modified Index Flood Method

Estimate the design flow using the Modified Index Flood Method (1995 version) for both a northern and a southern Ontario type watershed. Details of the watersheds are shown in the figure below.

Watershed Characteristics



Given

The watershed has a total area of 187 km^2 and is comprised of areas with characteristics of both northern (shield) and southern Ontario.

Northern (Shield) Type Basin

- Topographic maps and land classification maps show that this area is predominantly forested with shallow depth sandy soils.
- Underlying bedrock is widespread, either exposed at the surface or at shallow depths below the soil.
- The upper basin has five percent of its area covered by lakes and wetland areas. The lakes and wetlands are distributed evenly over the area.
- The basin area, A = 70 km^2, is predominantly of the Shield type.

Southern Type Basin

- A part of this area has agricultural land use. This portion of the watershed is of the Southern type and has mostly higher-runoff soils, such as silt and silty-clay.
- The lakes and wetlands are distributed evenly over the area.
- The basin area, A = 117 km^2, is predominantly of the Southern Type

Solution

- **Application for the Southern Ontario Type Basin**

CN Calculations

Land use Curve Number (CN) values were obtained by analysis of land classification maps and aerial photographs (Design Chart 1.09). This analysis is only required for the Southern type portion of the watershed. The soils data is summarized and the CN calculations are shown in the table below.

CN Calculation

Sub Area #	A (km^2)	Hydrologic Soil Group	CN	Σ(A * CN) (km^2)
1	9	BC	65	585
2	8	C	71	656
3	9	C	76	684
4	91	C	71	6461
Total	117			8386

$$\text{Average CN} = \frac{\Sigma\,(A * CN)}{\Sigma\,(A)} \tag{8.21}$$

$$= \frac{8386}{117} = 72$$

Watershed type: Southern

Watershed area: A = 117 km^2.

Main channel length: L = 27000 m, represents the longest flow path from the upper watershed boundary to the site.

Watershed slope: S_w = 0.0060, calculated using the 85/10 method.

Water storage area: A_d = 6.0 km^2, representing the total area of lakes and wetlands in the watershed sub-area.

Storage, (%) $= A_d / A * 100\%$ (8.23)
$= 6.0\ km^2 / 117\ km^2 = 5\%$

Soil/land Use Curve Number, CN = 72 (from the table above)

Base watershed class for the Southern type basin = 8.1 (CN = 72). (Design Chart 1.17)

Watershed Class Adjustments

Slope adjustment is estimated as +0.4, (for S = 0.006 m/m) (Design Chart 1.18)

Storage/detention adjustment is estimated as -0.6. (Design Chart 1.19)
(for % area lakes and wetlands = 5%)

Net adjustment, based on the algebraic sum of the above, is - 0.2

Net watershed Class = 8.1 - 0.2 = 7.9

Class coefficient, C = 1.79 (for Net Watershed Class = 7.9) (Design Chart 1.15)

$Q_{25} = C * A^{0.75}$ (8.20)
$= 1.79 * 117^{0.75} = 73\ m^3/s$

- **Application for the Northern Ontario (Shield) Type Basin**

Watershed type: Northern

Watershed area: $A = 70\ km^2$, from watershed mosaic

Water storage area: $A_d = 3.5\ km^2$, from watershed mosaic

Storage, % $= A_d/A * 100\%$
$= 3.5\ km^2 / 70\ km^2 * 100 = 5.0\%$

Watershed Base Class = 5.6. (Design Chart 1.16)

Location of storage: Well distributed throughout watershed.

Therefore, the storage is type B and the corresponding watershed class adjustment is zero.

Net watershed class = 5.6, the algebraic sum of the base class and storage location adjustment is zero

Class coefficient, C = 0.79; (for net watershed class = 5.6) (Design Chart 1.15)

Q_{25} = C * $A^{0.75}$ (Equation 8.20)
= 0.79 * $70^{0.75}$
= 19 m^3/s

Combine Watershed Subareas

$$\text{Overall watershed class} = \frac{\Sigma\,(A * \text{class})}{\Sigma\,(A)} = \frac{117 * 7.9 + 70 * 5.6}{187} = 7.0$$

Estimate 25-year flow using the total area of 187 km^2 and a base class of 7.0.

Class coefficient, C = 1.29 (Design Chart 1.15)

Q_{25} = C * $A^{0.75}$ (Equation 8.20)
= 1.29 * $187^{0.75}$ = 65 m^3/s

Apply Frequency Conversion Factor

Apply the frequency conversion factor, fcf, to convert results from the 25 year return period to the desired 50-year return period. Compare results for the total watershed area to each of the northern and southern components individually.

Northern Area, Q_{25} = 19 m^3/s; (for A = 70 km^2 and watershed class = 5.6)

Q_{50} = Q_{25} * fcf (Design Chart 1.15)
= 19 m^3/s * 1.13 = 22 m^3/s

Southern Area, Q_{25} = 73 m^3/s; (for A = 117 km^2 and watershed class = 7.9)

Q_{50} = Q_{25} * fcf (Design Chart 1.15)
= 73 m^3/s * 1.16 = 84 m^3/s

Total Watershed Area, Q_{25} = 65 m^3/s; (for A = 187 km^2 and watershed class = 7.0)

Q_{50} = Q_{25} * fcf (Design Chart 1.15)
= 65 m^3/s * 1.15 = 75 m^3/s

Note: The fcf value is interpolated between those for the northern and southern watersheds.

The Northern Ontario Hydrology Method

This method, developed by the Department of Civil Engineering, Queen's University of Kingston, Ontario for the MTO fulfils a need to provide realistic flow rates for design of water crossings across ungauged streams with small to medium watershed areas ($1\ km^2 < A < 100\ km^2$) in northern Ontario. Previous methods, based on rainfall only, are of questionable applicability to northern Ontario watersheds where inland lakes have a pronounced effect on the rainfall-runoff relationship. For smaller watersheds, the Rational Method is appropriate, and for larger watersheds, other statistical frequency analysis methods are appropriate.

This method is based on flood quantities estimated using probabilistic/statistical methods with data from 15 stream gauge stations across northern Ontario. The periods of record of these stream gauges ranges from 11 years to 31 years. The data from each station was checked for randomness, independence, stationarity and homogeneity to assess its validity. Then each data set was correlated to its watershed physiographic characteristics; watershed area, channel slope (and related parameters), watershed shape factor, area of lakes and wetlands, and CN number (SCS curve number). Results of this data correlation show that only watershed area, A, and area of lakes and wetlands, A_d, have a significant effect on flow rates. The type of watershed outlet (*normal* or *lake*) also affected peak stream flows. A series of equations were then developed through regression analysis to provide maximum daily discharge and maximum instantaneous (peak) discharge for the desired return period flow event. For details on the theory and development of this method, refer to the MTO publication: *Development of Hydrology Method for Medium-sized Watersheds in Northern Ontario* (1994).

This method is relatively easy to apply (see Example 8.9). It requires only four data entries: watershed area, A (km^2); area of storage (lakes and wetlands), A_d (km^2); desired return period of flow event, T (years); and type of watershed outlet (*normal* or *lake*). A series of equations (Equations 8.23 to 8.29) are solved to provide the desired maximum daily discharge (m^3/s) or maximum instantaneous (peak) discharge (m^3/s).

The above referenced MTO publication lists three limitations to the use of this method:

- the basin must be within the Canadian Shield;
- the drainage area, A, must be between $1\ km^2$ and $100\ km^2$; and
- for lake outlet situations, A_d / A must be at least 6%.

This method has a relative standard error in the order of 30%, however error in individual applications may exceed this. The user should be aware of such limitations and error.

Example 8.9: Estimating Flow Using the Northern Ontario Hydrology Method

Required

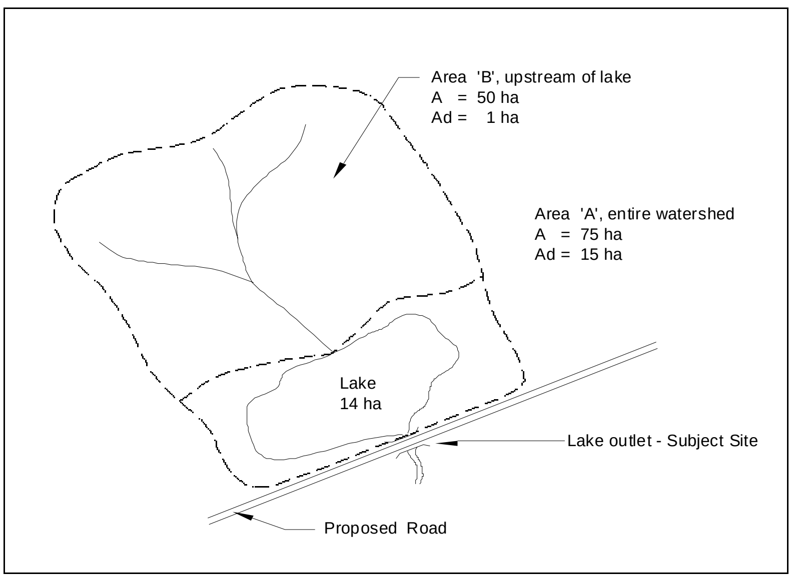
Estimate the design flow corresponding to a 25-year return period for an ungauged watershed in northern Ontario.

Given

The total watershed area is 7.5 km^2. Ponds and wetlands cover approximately 0.1 km^2 and are evenly distributed through the watershed. Also a 1.4 km^2 lake is located immediately upstream of the site. Therefore, the total area of lakes and wetlands is approximately 1.5 km^2. For purposes of illustration and comparison, two cases, A and B, are analyzed. Case A includes the entire watershed of 7.5 km^2 with 1.5 km^2 of lakes and wetlands. Case B includes only the area upstream from the lake with a total area of 5.0 km^2 and wetlands covering 0.1 km^2.

The watershed map is shown in the figure below.

Watershed Plan



Watershed Characteristics

Case	Total Area A (km^2)	Area Lakes and Wetlands A_d (km^2)	Proportion of Storage A_d / A
A	7.5	1.5	0.20

B	5.0	0.1	0.02

Solution

1. Calculate Mean Annual Discharge, Q_m

$Q_m = 0.170 * A^{1.06} (1 - A_d/A)^{2.07}$ (8.23)

Case A, Q_m $= 0.170 * 7.5^{1.06} (1 - 1.5/7.5)^{2.07}$
$= 0.907\ m^3/s$

Case B, Q_m $= 0.170 * 5.0^{1.06} (1 - 0.1/5.0)^{2.07}$
$= 0.897\ m^3/s$

2. Calculate Coefficient of Discharge, C_v

$C_v = 0.502 * (1 - A_d/A)^{1.85}$ (8.24)

Case A, C_v $= 0.502 * (1 - 1.5/7.5)^{1.85}$
$= 0.332$

Case B, C_v $= 0.502 * (1 - 0.1/5.0)^{1.85}$
$= 0.484$

3. Calculate Coefficient of Skew, C_s

$C_s = -2.52 + [3.73 * (1 - A_d/A)]$ (8.25)

If $C_s < 0.5$, use 0.5

Case A, C_s $= -2.52 + [3.73 * (1 - 1.5/7.5)]$
$= 0.46$, use 0.5

Case B, C_s $= -2.52 + [3.73 * (1 - 0.1/5.0)]$
$= 1.14$

4. Determine Frequency Factor, k (Design Chart 1.20)

Return period, T = 25 years

Case A, k = 1.93, where $C_s = 0.5$

Case B, k = 2.04, where $C_s = 1.14$

5. Calculate Maximum Daily Discharge, Q_t

$$Q_t = Q_m * (1 + k * C_v) \tag{8.26}$$

Case A, Q_t $= 0.907 * (1 + 1.93 * 0.332)$
$= 1.49\ m^3/s$

Case B, Q_t $= 0.897 * (1 + 2.04 * 0.484)$
$= 1.78\ m^3/s$

6. Calculate Peaking Factor, P_f

Case A, for the lake outlet situation:

$$\begin{aligned} P_f &= 1 + e^{[-22 * (Ad/A - 0.06)]} \\ &= 1 + e^{[-22 * (1.5/7.5 - 0.06)]} \\ &= 1.05 \end{aligned} \tag{8.27}$$

Case B, for the normal outlet situation:

$$\begin{aligned} P_f &= 1 + 6 * A^{-0.36} * e^{[-18 * (Ad/A)]} \\ &= 1 + 6 * 5.0^{-0.36} * e^{[-18 * (0.1/5.0)]} \\ &= 3.35 \end{aligned} \tag{8.28}$$

7. Calculate Maximum Instantaneous Discharge, Q_{pt}

$$Q_{pt} = Q_t * P_f \tag{8.29}$$

Case A, Q_{pt} $= 1.49 * 1.05$
$= 1.56\ m^3/s$

Case B, Q_{pt} $= 1.78 * 3.35$
$= 5.96\ m^3/s$

Discussion

Note that the flow from the case B watershed is nearly four times that from the case A watershed. This is due to greater lake storage in the case A situation. The flow peak is effectively damped by the lake.

Single Station Frequency Analysis

Statistical frequency analysis is one of the basic approaches available to determine the magnitude of a design flood. With this method, annual floods recorded at a stream gauging station are statistically correlated to provide a reasonably accurate means of estimating a design discharge.

The method involves interpretation of past stream flow data and derivation of a probability of occurrence by fitting a data series into a theoretical probability distribution. The discharge corresponding to the required design frequency may then be read from the distribution function curve.

The major limitations in using single station frequency analysis are the availability of a suitable length of stream flow records and quality of the data available. A relatively short period may represent a non-typical wet or dry period that may not include any major floods. The accuracy of this method increases with the number of years of record.

Data for certain years may be missing as a result of discontinuous (broken) or incomplete records. Broken records arise as a result of a discontinuation of station gauges; while incomplete records arise as a result of gauges becoming disabled, during for example, a very severe historical event. Both type of missing data should be corrected based on procedures outlined by the U.S. Interagency Advisory Committee (1982).

Outliers may also exist in the stream flow record. Outliers are data values that depart significantly from the main trend of the data. The presences of outliers makes it difficult to fit the data to a distribution. Procedures to deal with outliers is described by the U.S. Interagency Advisory Committee (1982), Environment Canada (Pilon et al, 1985).

Stream flow data is available from Water Survey of Canada. Flow data records are reported in the form of daily averages and maxima, and instantaneous maxima over a calendar year.

Two basic types of stream flow data can be used in frequency analysis: the annual maximum and partial duration series. The partial series consist of all events above a specified magnitude of flow. The annual maximum series is most commonly used.

Common Probability Distributions

A number of probability distributions are used for hydrologic frequency analysis including:

- Normal;
- Lognormal;
- Three-parameter lognormal;
- Extreme value Type I (Gumbel or exponential);
- Extreme value Type 2;
- Extreme value Type 3;
- Pearson Type 3; and
- Log Pearson Type 3.

A detailed review of the methods can be found in one of the following references:

- *Handbook of Applied Hydrology* (Chow, 1964);
- *Probability and Statistics in Hydrology* (Yevjevich, 1972);
- *Frequency and Risk analysis in Hydrology* (Kite, 1977).

Several computer programs have been developed to fit those distributions. The Consolidated Frequency Analysis (Pillon et al., 1985) is a commonly used one, which provides the analysis and plots the appropriate distribution. A brief description of this program is given in Appendix 8A.

An approximate method can be used in the absence of the above computer program. With this method, data can be plotted on the appropriate probability paper to determine the type of distribution, as follows:

- The first process involves ranking a flow series in ascending order;
- Calculate a return period, T_r, using the Wiebull equation for the corresponding discharge:

$$T_r = (n + 1)/m \tag{8.30}$$

where:

n = number of years of record
m = rank order of flood flow rate (largest = 1)

The return period may be plotted versus the corresponding discharge on probability paper.

- A linear trend on the probability paper would indicate that the data best fits the selected distribution.
- The peak flow, Q_p, for the return period, T_r, may be read directly from the probability curve if T_r is shorter than the period of record. If T_r is longer, the curve may be extrapolated to twice the length of records with reasonable accuracy, provided that the curve is reasonably straight.

Example 8.10: Consolidated Frequency Analysis (CFA)

Required

Determine the design flow rates for the 2, 10, 20, 50 and 100-year return periods based on historical records of continuous stream flow data, using Consolidated Frequency Analysis (CFA).

Given

The watershed area above the site of interest is approximately 78 km^2. The present land use is natural forest and agricultural and no significant changes in land use are foreseen. A stream flow gauging station is located in the immediate vicinity of the subject site. The stream flow has been gauged by Water Survey Canada, with continuous records from 1971 to 1994, a period of 24 years. The table below provides stream flow data for this gauge site:

Stream Flow Data

Year	Month of observation	Flow (m^3/s) Maximum Daily	Flow (m^3/s) Max. Instantaneous
1971	02	60	70
1972	05	45	
1973	04	39	
1974	04	60	
1975	05	62	
1976	03	72	85
1977	02	55	
1978	04	58	
1979	05	97	115
1980	05	32	
1981	09	62	
1982	04	71	
1983	02	51	
1984	02	58	
1985	03	64	
1986	05	87	
1987	04	110	125
1988	04	35	
1989	03	64	
1990	06	67	
1991	05	41	
1992	07	55	
1993	04	51	
1994	05	58	

Solution

Data Analysis Tests

Several tests were performed on the recorded data to ensure that no inherent biases exist.

Spearman Test for Independence

Spearman rank order serial correlation coefficient	= - 0.110
Degree of freedom	= 21.0
Corresponding to Students t	= - 0.507
Critical t-value at 5% level	= 1.721 (not significant)
Critical t-value at 1% level	= 2.518 (not significant)

Interpretation At the 5% level of significance, the correlation is not significantly different from zero. That is, the data do not display significant dependence. A subsequent data value does not tend to depend on the preceding value. The above results therefore indicate that this sample data exhibits a reasonable level of independence of data elements.

Spearman Test for Trend

Spearman rank order serial correlation coefficient	= - 0.008
Degree of freedom	= 22
Corresponding to Students t	= - 0.037
Critical t-value at 5% level	= - 2.074 (not significant)
Critical t-value at 1% level	= - 2.819 (not significant)

Interpretation At 5% level of significance, the correlation is not significantly different from zero. That is, the data do not display significant trend. Therefore, the characteristics of the data are not gradually changing over an extended period of time.

Run Test for General Randomness

Number of runs above and below the median	= 12
Number of observations above the median (N_1)	= 12
Number of observations below the median (N_2)	= 12
Range at 5% level of significance:	= 8 to 18 (not significant)

Interpretation The null hypothesis is that the data are random. At the 5% level of significance, the null hypothesis cannot be rejected. That is, the sample is significantly random. The frequency that sequential data values vary above and below the median value of the data set is checked. By this means, short and long cycle oscillations would be identified and rejected by this test. Therefore, it may be concluded from the above results that this data set is random.

Mann - Whitney Split Sample Test

Data sample split by time spans:
sub-sample 1: sample size = 12
sub-sample 2: sample size = 12
Mann-Whitney U = 71.5
Critical U-value at 5% significance level = 42.0
Critical U-value at 1% significance level = 31.0

Interpretation The null hypothesis is that there is no location difference between the two samples. At the 5% level of significance, there is no significant location difference between the two samples. That is, they appear to be from the same population. The results of this analysis reflect that the data elements have resulted from a similar phenomenon, are from the same watershed and do not reflect a significant change in the land use.

In conclusion, these statistical tests indicate that the data set reasonably meets the requirements of a probability distribution.

Statistical Analysis

Flow Frequency Distribution The period of years covered by the above data is sufficient to perform statistical analyses to determine river flows of various storms. However, discretion should be used in applying the results for the storm flows of longer return period because the accuracy is reduced. For purposes of comparison, four different frequency distributions are investigated. These distributions are the Gumbel, Three Parameter Log Normal (3-PLN), Log Pearson and Wakeby. The results are presented below:

Flow Frequency Estimates (Discharge, m^3/s)

Return Period (years)	Gumbel	3-PLN	Log Pearson	Wakeby
2	57.8	57.9	58.1	58.7
5	74.0	74.1	74.3	70.5
10	84.5	84.5	84.6	82.2
20	94.4	94.2	94.2	96.9
50	107	107	106	123
100	116	116	115	148

In assessing the above frequency plots to select a suitable one, the following relationships are assessed:

Coefficient of variation (C_v) is the standard deviation divided by the mean. The estimated values of C_v for the above frequency distributions are listed below:

Gumbel (GEV)	0.162
3-PLN	0.073
Log Pearson	0.072
Wakeby	0.162

The above results show that 3-PLN and Log Pearson show lower values for C_v, meaning that they have a higher level of reliability relative to the other methods.

Coefficient of skew (C_s), when equal to zero, indicates perfect symmetry in the distribution of data. The estimated values of C_s for the above frequency distributions are listed below:

Gumbel	0.154
3-PLN	0.007
Log Pearson	0.019
Wakeby	0.154

The above results show that the 3-PLN is a superior choice for the frequency distribution compared with the others as it provides the lowest C_s value.

Coefficient of kurtosis (C_k), when equal to -3.0, indicates a theoretical normal distribution. The estimated values of C_k for the above frequency distributions are listed below:

Gumbel	0.283
3-PLN	-3.883
Log Pearson	3.885
Wakeby	0.283

Comparing the above values for C_k, 3-PLN, seems to be the preferred choice for the frequency distribution.

Review of the above tests and visual examination of the data fits produced shows that 3-PLN is the best fit and therefore is adopted for further work. This is also the recommended frequency distribution for Ontario (*Regional Flood Frequency Analysis For Ontario Streams*; November, Environment Canada,1985).

The plot of the 3-PLN analysis carried out is shown below.

Maximum Instantaneous Flow Estimates The records of the stream flow reported in

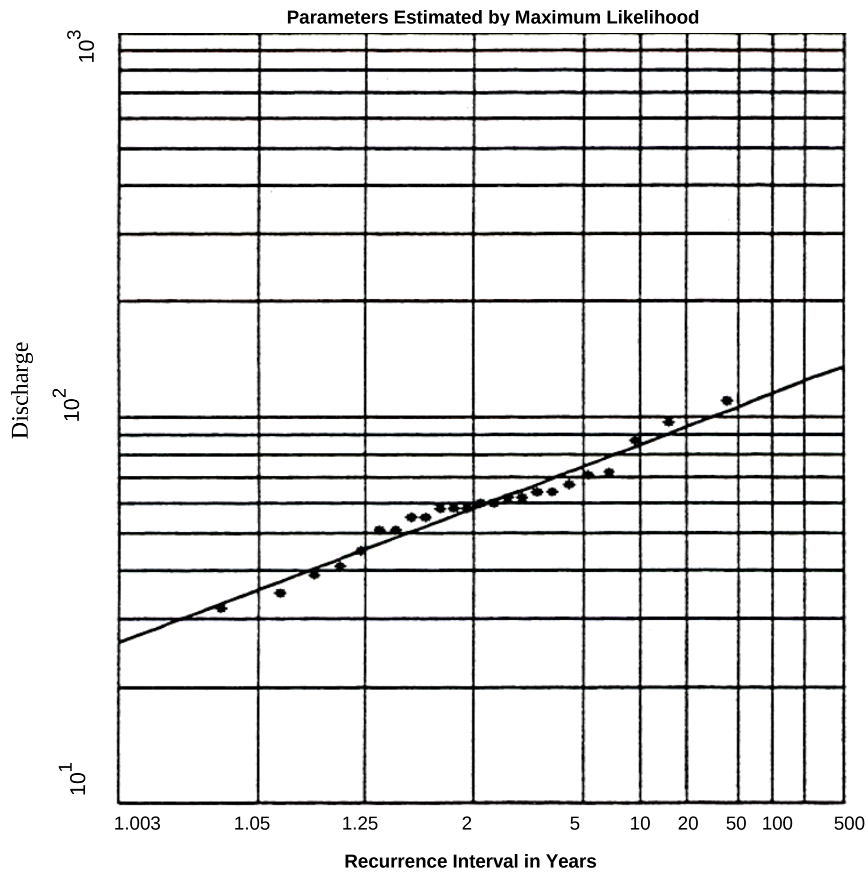
the stream flow data table above show four years, 1971, 1976, 1979 and 1987 with data for both maximum daily and maximum instantaneous flows. Comparing those values shows that the ratio is in the order of 1.1 : 1.0. Accordingly, the results of 3-PLN frequency distribution are adjusted by multiplying by 1.1 as:

Estimated Return Period Flows

Return Period (years)	Maximum Instantaneous Flow Discharge (m^3/s)
2	63.7
10	81.5
20	93.0
50	117.7
100	127.6

The above estimates of return period flow are recommended for the design of the water crossing at this location. Other return period flows may be estimated by interpolation between the values reported.

Figure 8.10: Plot of 3-PLN Frequency Distribution Analysis



Transposition of Flood Discharges

Sometimes it is necessary to transpose a discharge from a gauging station to another point on the same stream or to an adjacent basin where the discharge is unknown. If the basins have similar characteristic, instantaneous peak discharges can be transposed directly using the expression

$$Q_2 = Q_1 \left(\frac{A_2}{A_1} \right)^{0.75} \tag{8.31}$$

Where:

- Q_1 = known peak discharge
- Q_2 = unknown peak discharge
- A_1 = known basin area
- A_2 = unknown basin area

This expression is based on the modified index flood method. If the basins have significantly different hydrologic characteristics, it would be preferable to use the modified index flood method directly, possibly using the transposed figure as a check.

Where two or more gauging stations are available in a reasonably homogeneous watershed, the discharge corresponding to a given frequency at each station can be plotted on logarithmic paper and the required discharge interpolated or extrapolated, within reasonable limits.

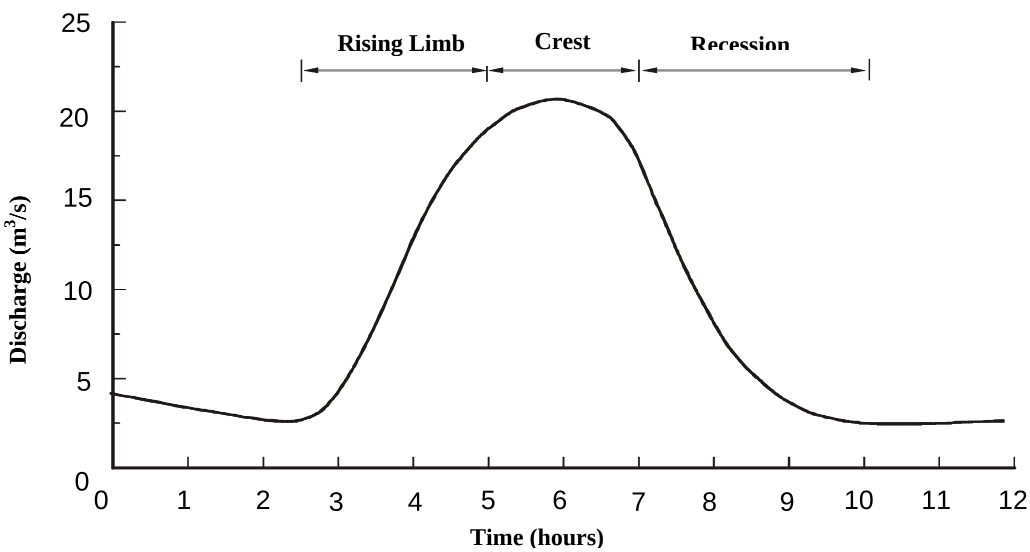
Hydrograph Methods

A hydrograph is a graph showing discharge versus time at a given point in a stream. It is as a result of the runoff processes comprising of overland flow, interflow and baseflow that are part of the hydrologic cycle.

The shape of a hydrograph is a reflection of the physical and meteorological conditions in a watershed. Figure 8.11 illustrates a typical hydrograph and the definitions of the standard parameters used to describe its shape.

A hydrograph consists of three parts, the rising limb, the crest and the recession limb. The shape of the rising limb is determined by the basin physiographic features, and the rainfall characteristics, such as duration, uniformity of distribution over the basin, and intensity. The crest extends between the inflection point of the rising and recession limbs. The peak flow rate, which occurs within the crest portion, generally indicates that all areas of the watershed are contributing flow. Theoretically, if a constant rainfall intensity prevails in the watershed, a constant peak outflow would continue. This often happens in long duration storms over relatively small watersheds where the time of concentration, t_c, is relatively short.

Figure 8.11: A Typical Hydrograph



Baseflow Separation

A stream flow hydrograph is made up of two parts: direct runoff (excess rainfall) and base flow (from groundwater). Separating the base flow portion is essential to assess the low flow characteristics of a stream.

In separating baseflow from direct runoff, the following approaches are available:

X straight line approach, which involves joining the beginning of the hydrograph (point A in Figure 8.12) to the end of the hydrograph at D;

X extend the recession curve for the preceding storm to point B;

X estimate point C to represent N (days) after the peak with the formula:

$$N = 0.8\ A^{0.2} \qquad (8.32)$$

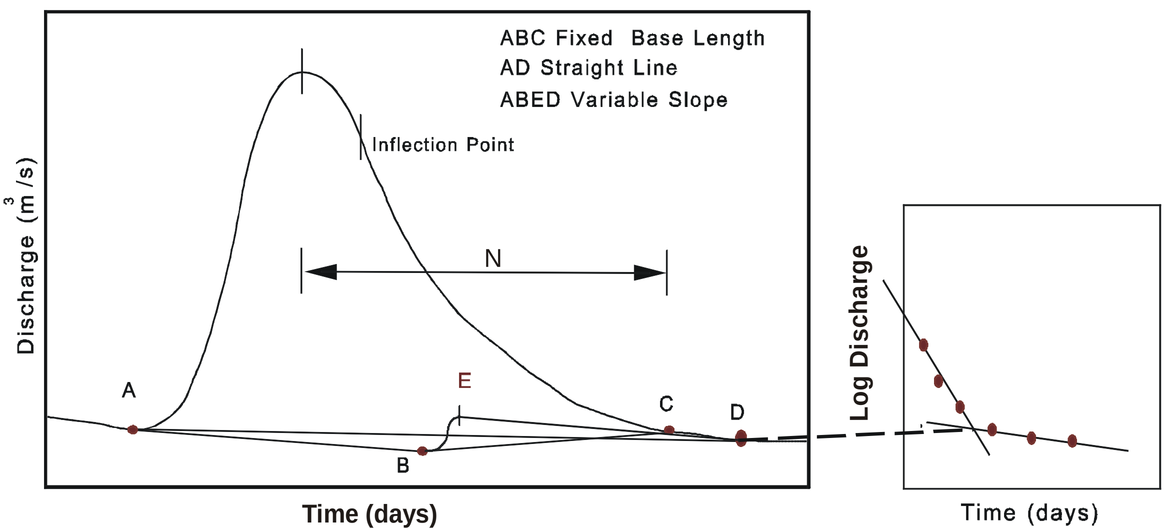
where:

A = area of watershed, km^2

and join point C to B to establish the separation.

X connect the slope of the recession limb of the hydrograph with the slope of the preceding one with a smooth curve as shown in Figure 8.12.

Figure 8.12: Methods of Base Flow Separation



(FHWA, 1980)

The volume below any of the lines ABC, ABED or AD, depending on the method selected, is considered to be the groundwater contribution.

Example 8.11: Estimation of Base flow

Required

For the hydrograph given in the table below, separate the base flow from the total runoff using three methods of base flow separation illustrated in Figure 8.12.

Given

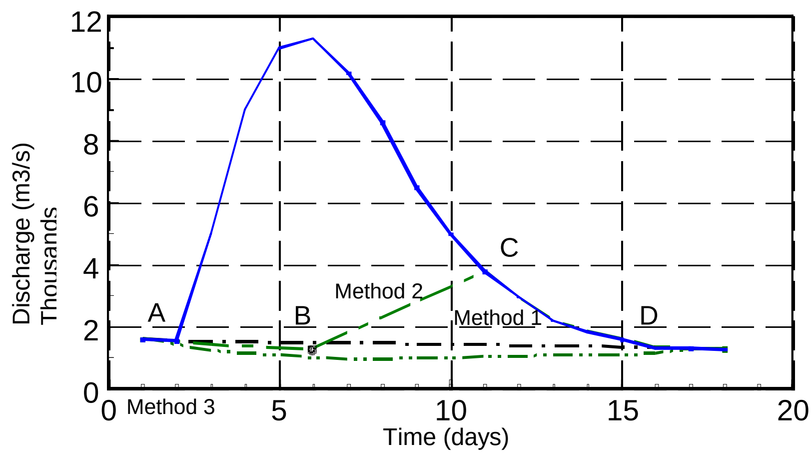
The area of the drainage site = 6,200 km^2.

Time	Total Flow (m^3/s)	Time	Total Flow (m^3/s)	Time	Total Flow (m^3/s)	Time	Total Flow (m^3/s)
0	0.1	20	117.2	40	21.5	60	10.8
2	1.2	22	94.9	42	20.0	62	10.2
4	4.9	24	73.5	44	18.6	64	9.8
6	24.3	26	59.2	46	17.2	66	9.3
8	150.7	28	48.2	48	16.0	68	8.9
10	196.2	30	40.0	50	14.8	70	8.5
12	197.2	32	34.5	52	13.8	72	8.1
14	179.5	34	30.0	54	12.8	74	7.7
16	160.9	36	26.3	56	11.9	76	7.3
18	139.5	38	23.3	58	11.4	78	7.0
						80	6.6

Solution:

Plot the hydrograph data
Determine the ordinates of the base flow hydrograph:

Method 1 Select point A, the beginning of direct runoff and point B, the end of direct runoff, by judgement. Join the two points with a straight line.



Method 2 Extend the recession curve preceding the storm up to point B below the peak. Compute point C as follows:

$$N = 0.8\,A^{0.2} \qquad (8.32)$$
$$= 0.8\,(6200)^{0.2} = 4.6 \text{ days (say 5 days)}$$

Join point B to point C with a straight line.

Method 3 Connect the slopes of the recession limbs with a smooth curve

Subtract the base flow from the hydrograph to define the direct runoff hydrograph. The ordinates of the direct runoff hydrograph resulting from the three methods are given in the table below.

Direct Runoff Ordinates

Time (days)	Direct runoff (m^3/s)		
	Method1	Method2	Method3
1	0	0	0
2	0	0	100
3	3,466	3,550	3,750
4	7,481	7,600	7,850
5	9,497	9,650	9,900
6	9,813	10,000	10,300
7	8,729	8,400	9,250
8	7,144	6,300	7,630
9	5,060	3,700	5,510
10	3,576	1,700	4,000
11	2,391	0	2,760
12	1,557	0	1,885
13	823	0	1,115
14	489	0	760
15	254	0	500
16	0	0	180
17	0	0	0
18	0	0	0

The Unit Hydrograph

The unit hydrograph method is one of the traditional methods used in flood estimation to specify a watershed response to a rainfall input. The method involves deriving a unit discharge relation and using it to evaluate the watershed response to other storms with other durations. The UH technique was developed by Sherman (1932).

The unit hydrograph (UH) of a drainage basin is defined as the hydrograph resulting from a unit depth of direct runoff generated from a uniform excess rainfall rate uniformly distributed over the basin area during a specified period of time. The traditional approach is to use one inch as a unit of runoff. In SI units, the corresponding runoff of 1 mm may be inconvenient, therefore, 25 mm can be used. This should be clearly indicated when applied. The time period is selected according to the basin response time, for example a1 hour or 4 hour response time results in unit hydrographs referred to as a 1-hour UH or 4-hour UH.

The concept of the UH is based on the idea that, similar storms will produce similar flood hydrographs because the characteristics of a given watershed remain relatively constant.

Hydrographs for larger and smaller rainfalls can be derived from the unit hydrographs by directly proportioning its ordinates upwards or downwards. For example, a storm generating 3 mm of direct runoff will produce a hydrograph whose ordinates are 3 times those of a 1 mm unit hydrograph for the same watershed. This is illustrated in Figure 8.13 In a similar manner, when 25 mm of net precipitation occurs in each of two consecutive unit time durations, the resulting hydrograph will be the sum of two 25 mm hydrographs, with the second one delayed by one time unit. UH concepts are incorporated in several popular computer models for flow estimation. Information on the unit hydrograph is presented as part of the standard output of the models.

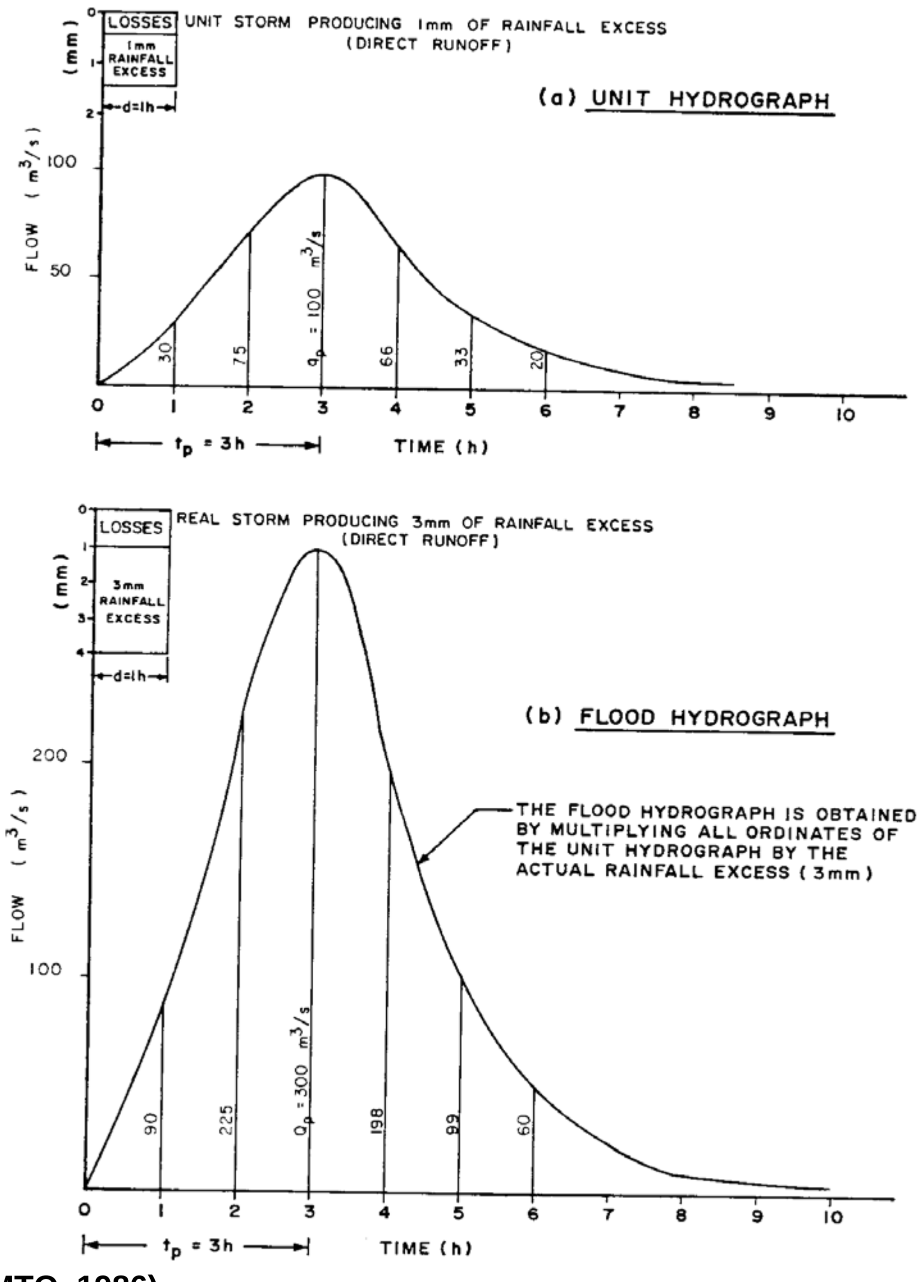
The unit hydrograph for a watershed can be developed using a number of methods. These methods depend on the type of data available. A unit hydrograph can be developed from hydrometeorlogical data (runoff, and rainfall) for gauged watersheds, or it can be generated using synthetic unit hydrographs based on watershed physiographic features (area, length, slope and shape).

Assumptions and Limitations

There are several assumptions and limitations to considered when applying the UH method:

- A hydrograph of runoff from a storm of variable intensity, or from a series of storms, may be constructed by adding the ordinates of a series of overlapping hydrographs, each resulting from an amount of storm runoff of unit duration.
- UH concept implies a linear relationship between rainfall input and direct runoff. However, watershed routing is non-linear, and this may create an underestimation of peak flow.
- The rainfall distribution is assumed to be the same for all storms of equal duration.

Figure 8.13: The Unit Hydrograph Concept



(MTO, 1986)

Development of the Unit Hydrograph for Gauged Watersheds

For gauged watershed streamflow data and precipitation data may be available from which the unit hydrograph can be developed. The method for deriving the unit hydrograph is described in example 8.14.

Example 8.12: Derivation of a Unit Hydrograph for Gauged Watersheds

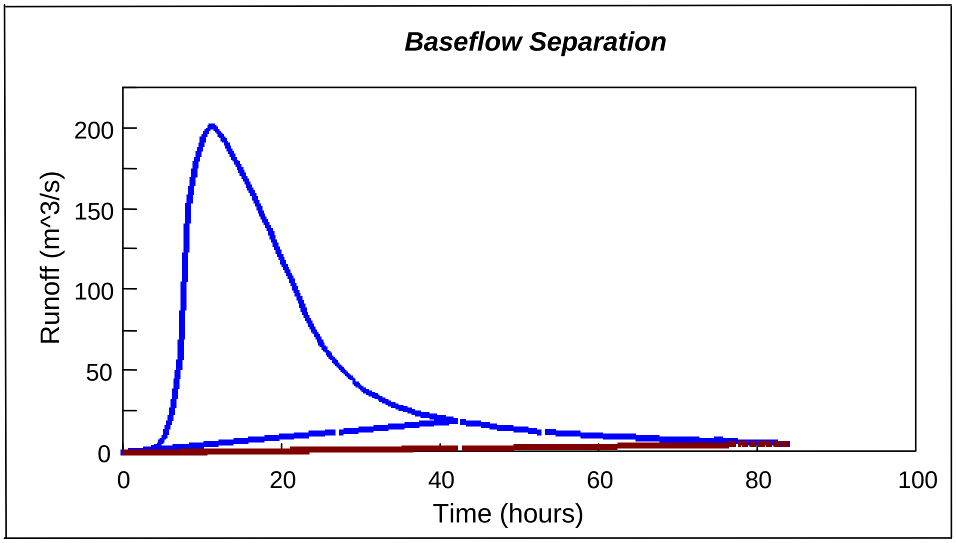
Required

Derive a unit hydrograph for a watershed, from the stream hydrograph given in Example 8.12 (values are also given in columns 1 and 2 of the summary table at the end of the example) and rainfall hyetograph shown below.

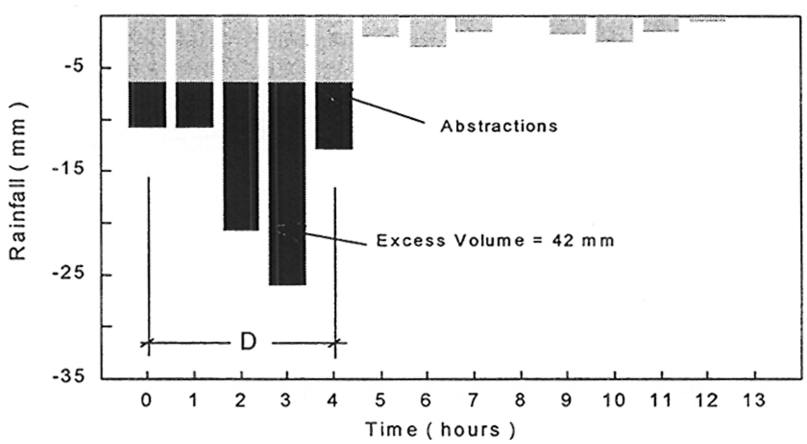
Given

Watershed area, A = 296 km^2

The stream flow hydrograph



Rainfall Hyetograph Solution



Solution

Separate the base flow from the direct runoff and calculate the total direct runoff. Since the base flow is a small part of the total stream flow, a simple straight line representation of base flow is used. Subtract the base flow from the total runoff hydrograph at equally spaced time intervals to define the ordinates, Q_j, of the direct runoff hydrograph as shown in table, columns 1,2 and 3.Compute the average depth of direct runoff, d_{ro}, over the watershed:

$$d_{ro} = t * EQ_j / A \tag{8.33}$$

where:

- EQ_j = Sum of direct runoff values Q_j, m^3/s
- A = Area of Watershed, m^2
- t = time interval, min

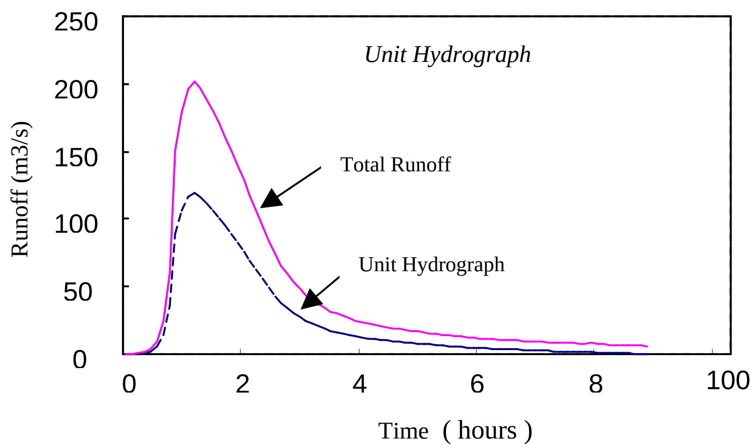
$$d_{ro} = 3600 * 3449 / (296 * 10^{-6})$$
$$= 0.042 \text{ m} = 42 \text{ mm}$$

Determine the effective duration of rainfall excess, D. Estimate the abstraction such that the volume of excess rainfall equals the average depth of direct runoff, d_{ro}. The corresponding duration is the specific duration of the unit hydrograph.

Compute the Unit Hydrograph ordinates for each time step as follows (column 5 in table):

$$U_j = (25 / d_{ro}) * Q_j$$
$$U_2 = (25 / 42) * 0.97$$
$$= .58 \text{ m}^3/\text{s}$$

The dashed line represents the 4-hour unit hydrograph per 25 mm (1 in.) of effective rainfall for the watershed.



Summary of Unit Hydrograph Ordinate Calculations

Time (h)	Total Flow (m^3/s)	Base Flow (m^3/s)	Direct Runoff Qj (m^3/s)	Unit Hydrograph Uj (m^3/s)
0	0.10	0.1	0.00	0.00
2	1.21	0.24	0.97	0.58
4	3.90	0.38	3.52	2.10
6	24.27	0.51	23.76	14.16
8	150.66	0.65	150.01	89.40
10	196.23	0.79	195.44	116.48
12	197.16	0.93	196.23	116.95
14	179.49	1.06	178.43	106.34
16	160.89	1.20	159.69	95.17
18	139.50	1.34	138.16	82.34
20	117.18	1.48	115.70	68.96
22	94.86	1.61	93.25	55.57
24	73.47	1.75	71.72	42.74
26	59.24	1.89	57.35	34.18
28	48.17	2.03	46.15	27.50
30	39.99	2.16	37.83	22.54
32	34.50	2.30	32.20	19.19
34	30.04	2.44	27.60	16.45
36	26.32	2.58	23.74	14.15
38	23.34	2.72	20.63	12.29
40	21.48	2.85	18.63	11.10
42	20.00	2.99	17.00	10.13
44	18.60	3.13	15.47	9.22
46	17.21	3.27	13.94	8.31
48	16.00	3.40	12.59	7.50
50	14.79	3.54	11.25	6.70
.....	.....	......	......	......
84	5.95	5.95	0.00	0.00
Sum			3449.06	

Development of the Unit Hydrograph for Ungauged Watersheds

For ungauged watersheds it is not possible to determine the unit hydrograph without the aid of theoretical and empirical formulas. These formulas are referred to as synthetic unit hydrographs . There are a number of methods by which to describe the unit hydrograph. The formulas may describe the unit hydrograph in terms of the shape and time to peak, or in terms of the timing of the unit hydrograph recession constant K. The time to peak and recession constant can be expressed as functions of the physiography of the watershed (area, slope, length, and width). The discussion below will focus on the later method for describing the unit hydrograph. It will cover methods for evaluating the time to peak and recession constant.

Time to Peak

The time to peak is a controlling factor in determining the peak flow. A short time to peak indicates a rapid hydrologic response.

There are several methods used in calculating the time to peak, t_p. One commonly used one is the HYMO method (USDA, 1973).

HYMO Method The US Department of Agriculture developed a three-parameter equation for watersheds with slopes less than 2%, and a two-parameter equation for watersheds with slopes greater than 2%.

Three-parameter equation:

$$t_p = 0.0086 * A^{0.422} * S^{-0.46} * (L/W)^{0.133} \quad (8.34)$$

where:

t_p = time to peak, h
A = drainage area, ha
S = slope, m/m
L/W = length/width, dimensionless

Two-parameter equation:

$$t_p = 0.016 * A^{0.31} * S^{-0.50} \quad (8.35)$$

where:

units are as given above

For urban watersheds, the time to peak is reduced significantly, and is usually approximated by taking 50% of the rural value.

Example 8.13: Calculation of Time to Peak

Required

Estimate the time to peak, t_p of the watershed in Example 8.5.

Given

The following are two scenarios to assess:

(i) S =.5%, width = 300m
(ii) S =3%

Solution

(i) Use Equation 8.34 (HYMO 3-parameter) for slope less than 2%. Substituting values of A, S, L, W and converting into appropriate units:

$$t_p = 0.0086 * 7^{0.422} * 0.005^{-0.46} * (210/300)^{0.133} \qquad (8.34)$$

$$t_p = 0.168 \text{ h}$$

(ii) Use Equation 8.35 (HYMO 2- parameter) for slope greater than 2%. Substituting values of A, S, L, W and converting into appropriate units:

$$t_p = 0.016\,(7)^{0.31}(0.03)^{-0.5} \qquad (8.35)$$

$$t_p = 0.14 \text{ h}$$

Recession Constant

The recession constant controls the shape of the hydrograph after the time to peak. Basically, two approaches are commonly used in determining the recession constant, namely HYMO and hydrograph methods.

HYMO Method The U.S. Department of Agriculture (1973) developed a two and three parameter equations for rural watersheds. For watersheds with slopes less than 2%, K can be determined from the following relationship:

$$K = 0.0095 * A^{0.231} * S^{-0.777} * (L/W)^{0.124} \qquad (8.36)$$

For other watersheds, K can be determined as follows:

$$K = 0.00316 * A^{0.24} * S^{-0.84} \quad (8.37)$$

where:

K	= recession constant, h
A	= drainage area, ha
S	= slope, m/m
L/W	= dimensionless

For urban areas, the value of K is estimated as 50% of the rural value for the same area.

Hydrograph Method This method utilizes the discharge/time relationship of a flood hydrograph.

The time variation of the flow per unit time after the peak (the recession limb) can be expressed as:

$$Q_t = Q_o * K_t \quad (8.38)$$

where:

Q_t	= discharge at time t after the peak, m^3/s
Q_o	= discharge at start of recession, m^3/s
K_t	= recession constant

This equation can be plotted as a straight line of log Q_t and time. K_t is the slope of the line.

$$\log K_t = (\log Q_t - \log Q_o)/(t_t - t_o) \quad (8.39)$$

Example 8.14: Calculation of Recession Constant Using Equations

Required

For the watershed data in Example 8.13, determine the recession constant K.

Given

Solve for the following two scenarios;

(i) S = 0.5%, width = 300m, predominantly urban
(ii) S = 3%

Solution

(i) Where the slope is less than 2%, use Equation 8.36 (HYMO 3-parameter method).

Substitute values of A, S, L, W and converting to appropriate units.

$$K = 0.0095 * A^{0.231} * S^{-0.777} * (L/W)^{0.124} \quad (8.36)$$

$$K = 0.0095 \; (4)^{0.231} \; (0.005)^{-0.777} \; (210/300)^{0.124}$$
$$= 0.77$$

Adjust the value of K for an urban watershed using

Urban K = Rural K * 0.5
= 0.77 * 0.5
= 0.385

(ii) Where the slope is greater than or equal to 3%, use Equation 8.37 (HYMO 2-parameter method)

$$K = 0.00316 * A^{0.24} * S^{-0.84} \quad (8.37)$$

$$K = 0.00316 \; (7)^{0.24} \; (0.03)^{-0.84}$$
$$= 0.084$$

Example 8.15: Calculation of Recession Constant Using Hydrograph Method

Required

Determine the recession constant of the discharge hydrograph given in the following table using the hydrograph method.

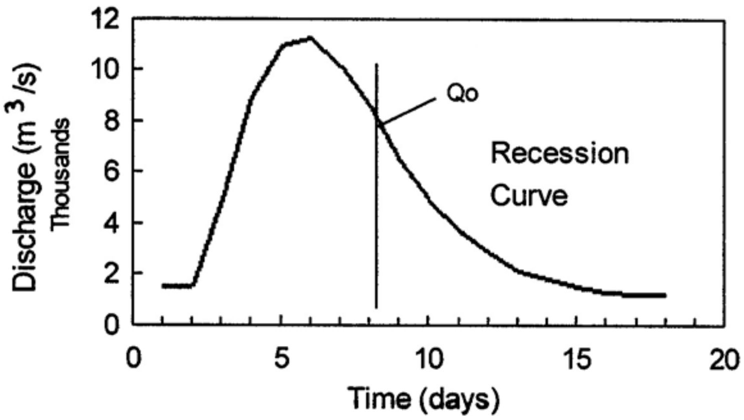
Given

Discharge Hydrograph

Time (days)	Flow (m^3/s) Q_t	Time (days)	Flow (m^3/s) Q_t
1	1600	10	5000
2	1550	11	3800
3	5000	12	2800
4	9000	13	2200
5	11100	14	1850
6	11300	15	1600
7	10200	16	1330
8	8600	17	1300
9	6500	18	1280

Solution

Plot Q_t versus time on a semi-logarithm graph. The resulting curve (below) can be approximated by three straight lines of decreasing slope representing the surface detention, interflow and groundwater components of the recession limb. The respective recession constants are given by the slope of each line.

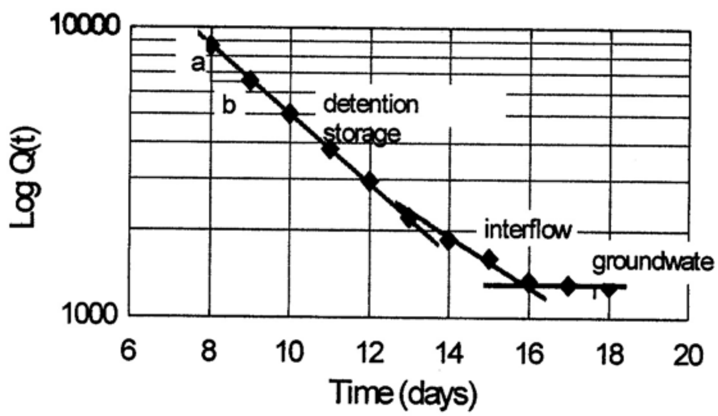


The slope of the line representing the detention storage recession curve is calculated as:

$$\log [K] = \frac{a}{b} = \frac{(\log[8600] - \log[6500])}{-1} \qquad (8.38)$$

$$= -0.128$$

$$K = 0.74$$



Hydrograph Simulation Methods

Hydrograph simulation methods and models are a more detailed way of estimating flows. Several computer based models are available such as HYMO, INTERHYMO, MIDASS, and SWMM to name a few. Refer to Table 8.5a for a more comprehensive list of models and to Appendix 8A for a more detailed description.

Application

The following are some advantages of using this method:

- X it provides an easy and quick way of undertaking a quantitative comparison of peak flow and runoff volumes for rural versus urban development conditions;
- X facilitates a lumped versus a distributed representation of a drainage area;
- X can be utilized for flow estimation in urbanized areas to examine various drainage management options, including storage and diversions; and
- X can simulate various land uses, times of concentration or significant soil type variations across a drainage area.

Application of any of the hydrologic/hydraulic simulation models to highway drainage design should only be undertaken with sound engineering judgement and a thorough understanding of its application and limitations.

Basic Structure

These models are mathematical analogies of rainfall/runoff transformation processes. Models such as HYMO (SCS, 1973) compute hydrographs for the sub-basins, route them through the channel system and combine them into a single hydrograph at the downstream end.

The basic process incorporated in runoff simulation models is illustrated in Figure 8.14 As illustrated, all simulation programs are composed of a group of subprograms which simulate the following hydrologic processes:

- X precipitation losses due to infiltration and percolation;
- X determination of rainfall excess; and
- X conversion of runoff into hydrographs.

In addition, most models consider the effects of storage as follows:

- X routing of a hydrograph through channels; and
- X routing of hydrographs through areas with significant storage.

Available Programs

Several hydrologic simulation methods are available to the analyst/designer. A brief summary of these programs is presented in Appendix A. They are broadly classified as either single or continuous event simulation programs.

Single Event Simulation

A single event model refers to a model that simulates the response of a watershed to a short duration (1 hour to a few days) design rainfall event. The design rainfall may be a physical event such as a historical storm or a synthetic storm based on a statistical analysis of recorded rainfall.

The main theoretical assumption of event analysis is that the simulated runoff is assigned the same frequency of occurrence as the precipitation. The deficiencies of this assumption can be summarized as follows.

X Precipitation and the final output hydrograph have many independent characteristics. Accordingly there is no direct correlation between any of the independent characteristic of a precipitation event and characteristic of the resulting hydrograph. For example, the antecedent moisture conditions in a watershed may be dry and a 1-year precipitation even may result in no runoff.

X The drainage system is a composite of various independent units. For example, in urban development situations, runoff is dependent on the major (overland flow) and minor (storm sewer) drainage systems. The frequency of runoff from these systems will vary. Therefore, the frequency of runoff cannot be defined based on the frequency of precipitation input alone.

Single event modelling is primarily used for estimating the peak flow rate. The anticident moisture conditions at the beginning of the simulation must to be assumed.

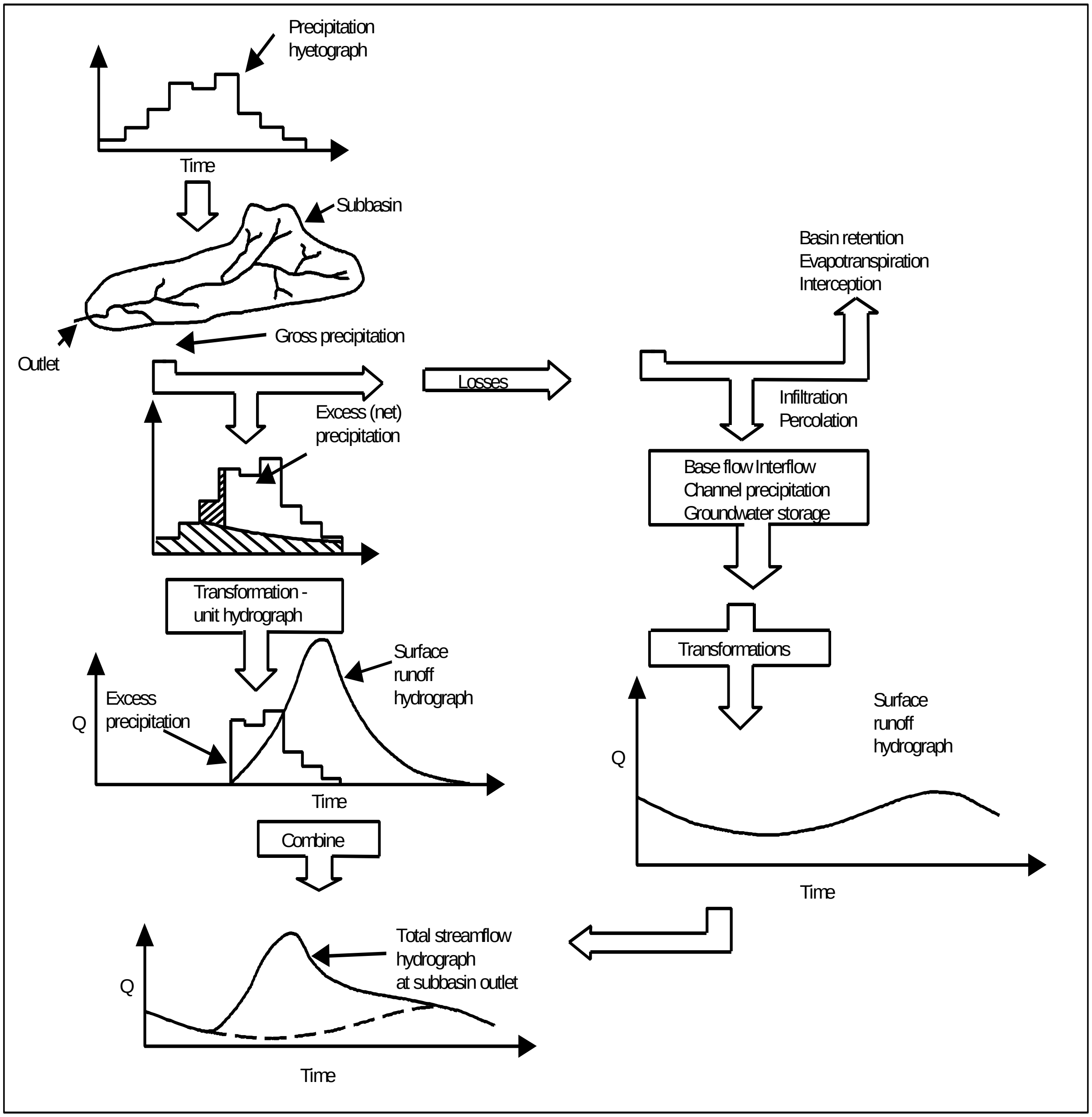
Continuous Simulation

A continuous model refers to a model that simulates the response of a watershed to precipitation using actual continuous rainfall data covering a long period of record (several days or months). With continuous simulation, antecedent moisture conditions are more representative of drainage conditions since they are not assumed but are a reflection of the sequence of wet and dry periods.

When results of failure of a drainage system represent a significant risk to life or property damage, continuous analysis may be warranted. However, continuous simulation is expensive and requires significant computer time to calibrate the model. Such calibration also requires specialized hydrologic expertise.

Continuous simulation is also preferred where low flow is of major interest. In these cases, infiltration and evaporation losses are sensitive and continuous simulation is able to account for the losses more closely than does single event simulation.

Figure 8.14: Typical Rainfall- Runoff Process of a Simulation Model



(Viessman, 1989)

Model Selection

In selecting a computer model for use, the following considerations noted in the publication *Guidelines for the Development and Use of Computer programs by Professional Engineers*, (APEO, 1986), should be considered:

- identification of the hydrologic processes involved (storage impacts, channel routing, land use changes, stormwater quality, etc);
- recognition of the accuracy involved (simple vs complex); and
- assess cost, time, skills and computer capability required.

The Professional Engineers of Ontario gives the following advice on the use of computer models:

- determine the exact nature of assistance the program provides;
- identify the theory on which the program is based;
- determine the limitations, assumptions, etc., that are included in the theory and program;
- check the validity of the program for the intended application; and
- verify that the results are correct for each application.

The purpose of this manual is not to provide a working knowledge of various analysis methods, but to provide some guidance on their selection and application. See Table 8.5 for information on model characteristics.

Calibration and Verification of Models

Calibration is the process whereby simulation is repeated for model input parameters until a good fit between measured and simulated values occurs. Verification on a different set of data (actual measured flow and rainfall data) determines the validation of the calibration process. These processes establish confidence in using the model.

Calibration is useful for estimating basin parameters which cannot be physically measured directly. Examples include time to peak, recession constant, infiltration parameters, directly connected impervious areas, Manning's roughness coefficient and depression storage.

Calibration and verification processes require large amounts of data. The measurement of data necessary to calibrate and verify a model is a time-consuming and expensive undertaking. If failure of a facility would increase the risk to life or property damage, then data collection for calibration should be considered.

It is good practice to perform a sensitivity analysis of the input parameters, especially if calibration and verification cannot be undertaken.

Sensitivity analyses are usually carried out on parameters which cannot be measured with sufficient accuracy. The sensitivity analyses will determine the variation in model output due to changes in input parameters. These analyses give the designer an understanding of the response of the hydrologic/ hydraulic process to model inputs and show the effects of inaccuracy of input data.

Flood Routing

Flood routing is a process that estimates the behaviour of a flood wave as it travels through the watershed to its outlet. In doing so, storage in natural depressions, lakes and reservoirs, main channels and valleys have the effect of reducing (attenuating) the magnitude of the peak flow and lengthening the time to peak to produce a time lag as it moves downstream. Figure 8.15 illustrates the typical effect of routing a flood hydrograph through watershed storage.

This concept is applied to flood control works. For example, in highway construction, the runoff peak flow and volume from the highway would usually increase due to increases in imperviousness. In order to compensate for those increases, a storage reservoir (detention pond) can be used to attenuate the flood wave. Similarly, channel storage can be utilized to produce similar effects.

The basis of flood routing between two points is derived from the fact that flow entering the first point must emerge at the second point, or it will remain in storage. This is the continuity principle and it can be expressed as:

$dS/dt = I - O$ (8.40)

where:

I = inflow rate, m^3/s
O = outflow rate, m^3/s
S = storage, m^3
dS/dt = change in storage with respect to time, m^3/s

A second relationship between storage and discharge is required in order to solve Equation 8.40. Figure 8.15 also illustrates schematically the inflow-outflow and storage relation of a flood wave.

Figure 8.15: The Inflow-Outflow Storage Relationship

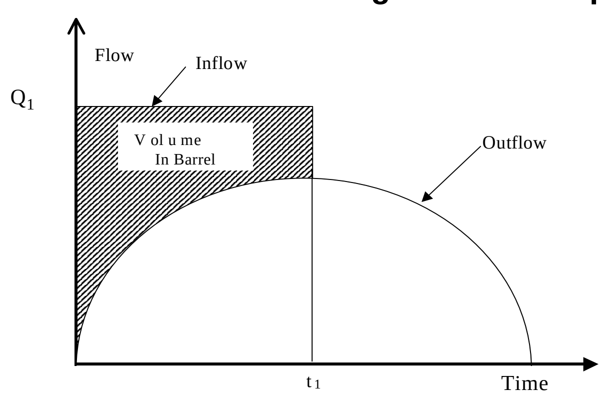


Table 8.5a: Hydraulic Computer Model Characteristics

Applications	Computer Program									
	MOBED	HYCHAN	FERNS	FLOW 1-D	HEC-2	HEC-6	HEC-15	WSPRO	EXTRAN	DWOPER
Flow Conditions:										
Steady	!	!	!	!	!	!	!	!	!	!
Unsteady	!	-	!	!	-	-	-	-	!	!
Gradually Varied	!	-	!	!	!	!	-	!	!	!
Rapidly Varied	-	-	-	-	-	-	-	-	-	-
Subcritical	!	!	!	!	!	!	!	!	!	!
Supercritical	!	!	!	!	!	-	!	!	!	!
Two Dimensional	-	-	-	-	-	-	-	-	-	-
Tractive Force	-	-	-	-	-	!	!	-	-	-
Energy	!	!	!	!	!	!	-	!	!	!
Momentum	-	-	-	-	-	-	-	-	-	-
Output:										!
Water Surface Profile	!	!	!	!	!	!	-	!	!	-
Velocity Profile	!	-	-	!	!	-	!	!	-	-
Ice	-	-	-	-	!	-	-	-	-	-
Cross Section	-	-	-	-	!	-	-	!	-	
Flow Distribution										
Options:										
Tributary Profile	-	-	!	!	!	!	-	!	!	!
Multiple Profile	-	-	!	!	!	-	-	!	-	-
Automatic Calibration	-	-	-	-	!	-	-	!	-	!
Bridge/Culverts	-	-	!	-	!	-	-	!	-	!

Table 8.5a: Hydrologic Computer Model Characteristics

Applications	Single Event					Continuous Event			
	HYMO	OTTHYMO	OTTSWMM	ILLUDAS	MIDUSS	HSP-F	QUALHYMO	STORM	SWMMIV
Land Use: Urban						!	!	!	!
Rural	!	!	!	!	!	!	!	!	!
Infiltration	!	!	!	!	!	!	!	!	!
Temperature			!			!	!	!	!
Evapotranspiration			!			!	!	!	!
Subsurface Flow						!	!		
Water Balance						!	!	!	!
Water Quality						!	!	!	!
Hydrograph Method	!	!	!	!	!	!	!	!	!
Routing:									
Watercourse\Channel	!	!	!	!	!	!	!		!
Reservoir	!	!	!		!	!	!		!
Water Quality			!			!	!	!	!
Major/Minor System			!						
Receiving Water						!	!		
Ontario Suitability	Y	Y	Y	Y	Y	Y	Y	Y	Y
Level of Effort	L	L	M	L	L	H	M	M	M
Data Requirements	M	M	H	M	M	H	M	M	H

Legend ! -suits application, Y-Yes, L-low, M-medium, H-high

Channel Routing

Figure 8.16 shows the changes to a flood hydrograph as it is routed along a channel from an upstream point.

The continuity equation, Equation 8.40 and a storage discharge relationship are used to evaluate the effect of attenuation and time delay of a flood wave due to the flood storage of a channel. By considering a sequence of inflows and outflows a time interval,)t, the continuity equation, Equation 8.40 can be expressed as:

$$½ (I_1 + I_2) - ½ (O_1 + O_2) = (S_1 - S_2) /)t \quad (8.41)$$

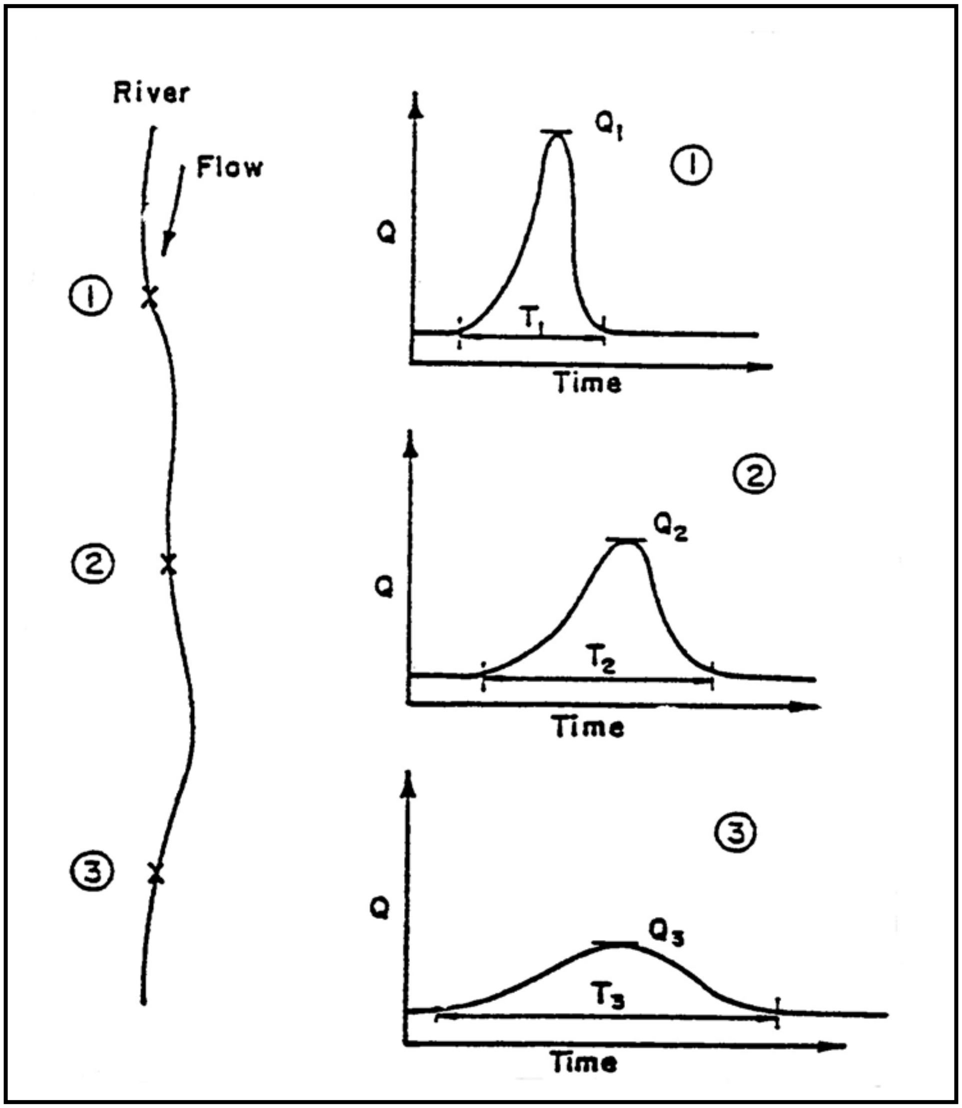
where:

Subscripts 1 and 2 denote values at the beginning and end of the time interval,)t (seconds).
I and O are inflows and outflows, m^3/s.
S is storage, m^3.

Figure 8.17 shows an increment of storage over an increment of time. The chosen time interval,)t, is called the routing period and must be sufficiently short so that the straight line assumption between the two points at the beginning and end of)t is reasonable. Further, if the routing period is too long, it is possible to miss the peak flow. However, the amount of computation may be excessive if)t is too short.

In channel routing the storage-discharge relationship expresses the storage that occurs within a channel reach in terms of both inflow and outflow, as in the Muskigum. Using this method the basic hydraulic equations can be solved rapidly and accurately with a calculator. However, channel routing routines are often included as a component of most computer based hydrologic methods.

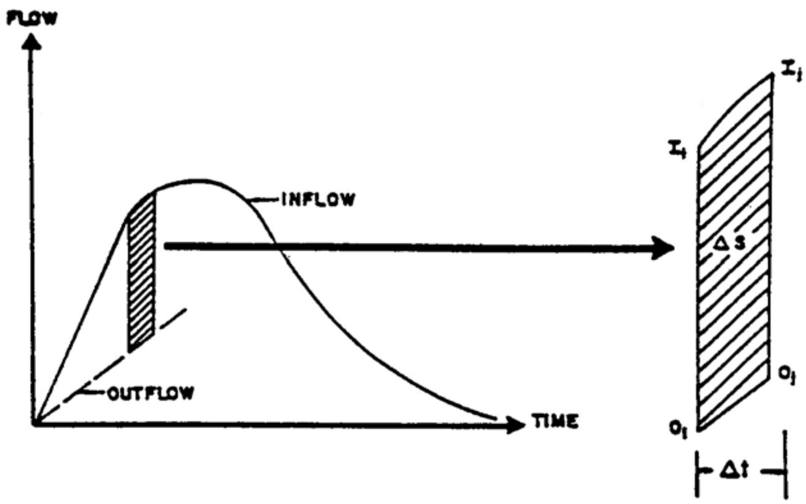
Figure 8.16: The Effects of Routing on a Flood Hydrograph



The effect of channel storage on a flood hydrograph as it moves through a river channel. Q_1, Q_2, Q_3, and T_1, T_2, and T_3 are the peak discharges and base length times for locations 1, 2 and 3.

(FHWA1980)

Figure 8.17: The Concept of Time and Storage Increment



Reservoir Routing

Similar to channel routing, reservoir routing transforms an inflow hydrograph as it moves from the inflow to the outflow end of a reservoir. There is one main difference, which is the storage-discharge relationship. In reservoir routing outflow occurs over a weir or through a pipe, which is independent of the reservoir inflow. Therefore, the storage-discharge relationship is expressed by a weir or orifice discharge equations, which is a function of water surface elevation. Since there is a relationship between the storage volume in a reservoir and water evaluation, or stage (the surface area of the reservoir is usually knows) it is possible to use a stage-discharge relationship instead of a storage-discharge equation when performing reservoir routing.

A simple case of reservoir routing is a detention pond receiving inflows at one end and discharging through a weir at the other end.

Several standard routing methods are used to compute the outflow hydrograph given an inflow hydrograph, using the continuity equation, Equation 8.40. For convenience, this equation is rearranged so that all known terms are on one side, as follows:

$$(I_1 + I_2) + (2S/)t + O)_1 = (2S/)t + O)_2 \qquad (8.42)$$

The two unknowns on the right hand side of Equation 8.42 are the increment of storage, S, and the outflow, O. For this reason both the continuity equation and storage-discharge are required to solve for the two unknowns.

Using a computer, the continuity equation and stage-storage functions are easily solved simultaneously to produce quick solutions. The mathematical details of this method are covered in most hydrology text books.

Low Flow Analysis

Low flows in streams typically represent the portion of rainfall that infiltrates into the soil and seeps into the streams through groundwater flow. It is usually referred to as the base flow portion (groundwater contribution) of a streamflow hydrograph.

Seasonal low flow may be required for the design of channels and culverts to address an environmental concern. The information may be needed to design temporary diversion works that replicate the low flow characteristics of the natural stream or select an appropriate construction period. It may also be necessary to maintain the low flows characteristics in a stream for a 3, 4 or 5-day period to facilitate fish migration. The following are the stream flow characteristics that may need to be maintained:

- low flow volume;
- low flow discharge;
- low flow stage; and
- low flow duration.

Low Flow Statistical Parameters

Low Flow Volume

This series may comprise of the minimum volume (V_t) for a given time interval (t) as illustrated in Figure 8.18. The corresponding low flow volume is usually taken as the minimum in the series, i.e.:

$$V = \text{minimum of } (V_1, V_2........V_n) \text{ for n time periods.} \tag{8.43}$$

Low Flow Discharge

Low flow discharge at a stream cross section may be defined as the minimum instantaneous or average flow during a specified time period.

Since low flow volume can be expressed as mean discharge for time period, t:

$$q_t = V_t / t \tag{8.44}$$

Then the design low flow discharge can be determined as:

$$Q = \text{minimum}\ (q_1, q_2 ....q_n) \tag{8.45}$$

Low Flow Stage

The low flow stage is the water level which corresponds to the low flow discharge. Except in situations where channelization of a stream has taken place, calculation of the low flow stage may not be necessary as it could be hydraulically derived from the flow or volume time series.

Low Flow Duration

Low flow duration, D, can be taken as the time for which the flows are smaller than a specified discharge, q, or water levels less than a specified stage, h, during a specified time period. This is graphically illustrated in Figure 8.18. For a given time period, the duration, D, is the sum of all time intervals that the flow (or stage) is below a specified rate (or level).

$$D = (d_1 + d_2 + ...d_n) \tag{8.46}$$

where:

d_i = independent duration i;
n = number of durations.

If the designer is interested in the longest of the independent durations, L, then this is obtained from the series as:

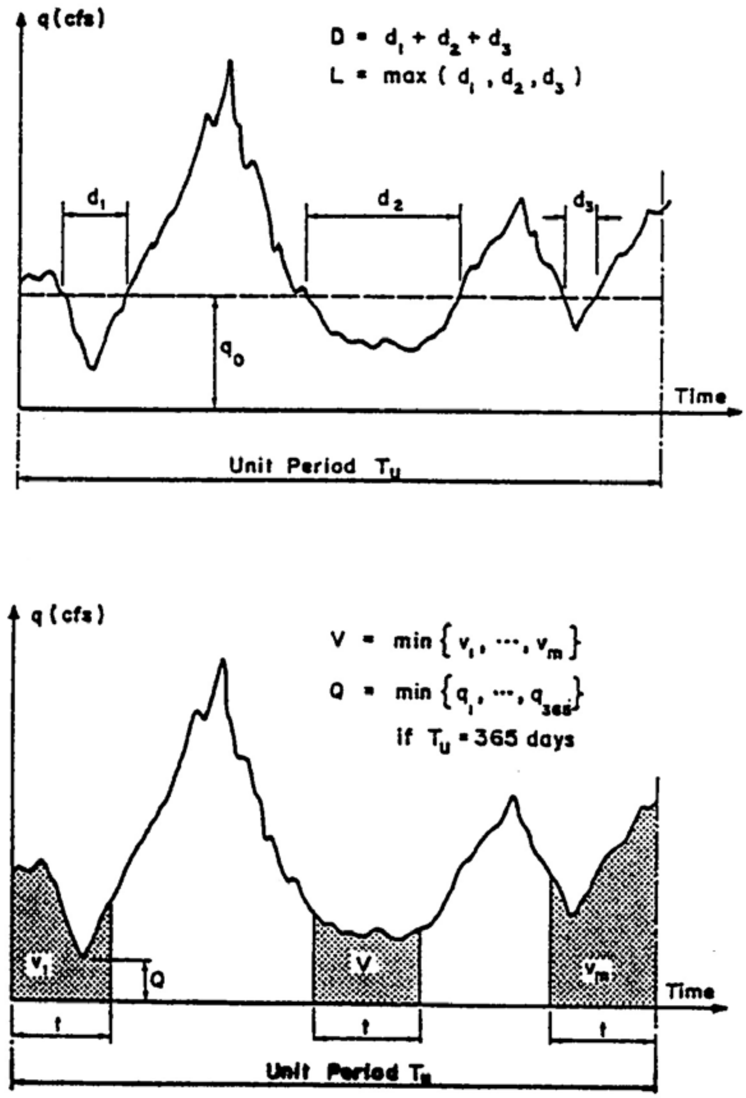
$$L = \text{maximum}\ (d_1, d_2, ...d_n) \tag{8.47}$$

Analysis of Low Flow Data

With the establishment of a time series characterising the low flow data, basic statistical parameters, such as mean values and frequency distributions, can be determined.

The technique of assigning a rank and probability to the low flow variable and plotting the data on a log normal paper to construct the best fitted line is an easy and recommended frequency distribution method as explained in the section on statistical frequency analysis.

Figure 8.18: Definition of Low Flow, Duration, Volume and Discharge



(FHWA, 1980)

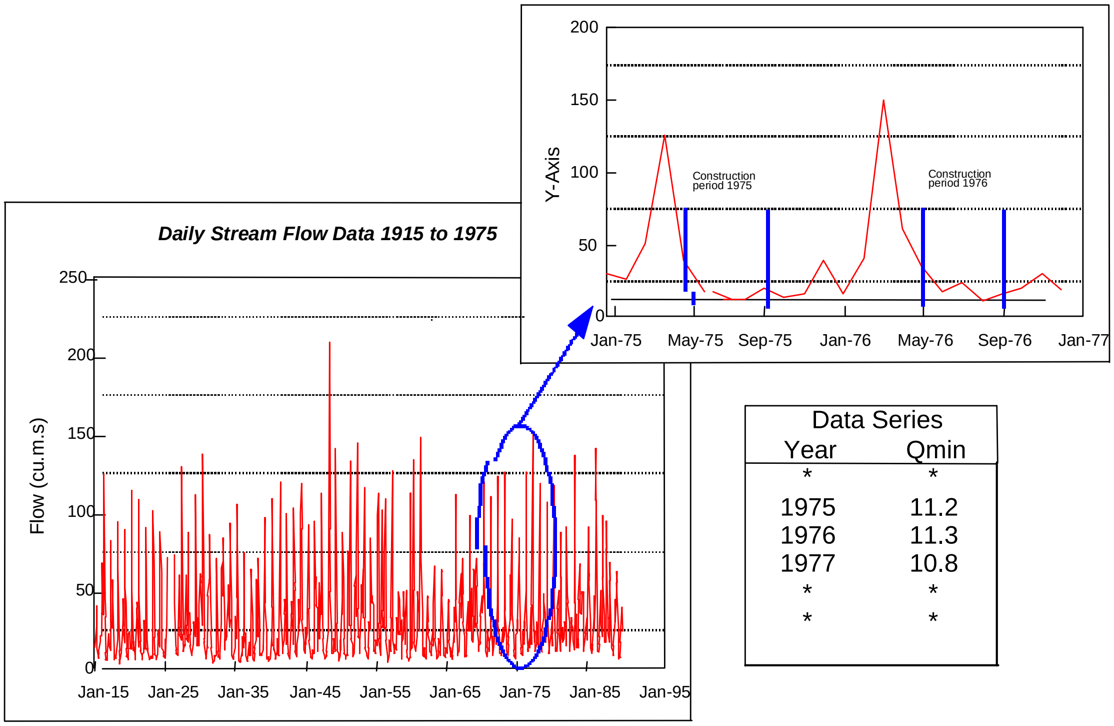
Example 8.16: Low Flow Analysis

Required

From stream flow records determine the low flow that has a 95% probability of not being exceeded, during the period from May to September (i.e. only 5% of the flows will be more than that value). What is the period of time in which this criterion is not satisfied (i.e. the flow will be less than that flow).

Given

The monthly stream flow is given in the following graph.



Data Series	
Year	Qmin
*	*
1975	11.2
1976	11.3
1977	10.8
*	*
*	*

Solution

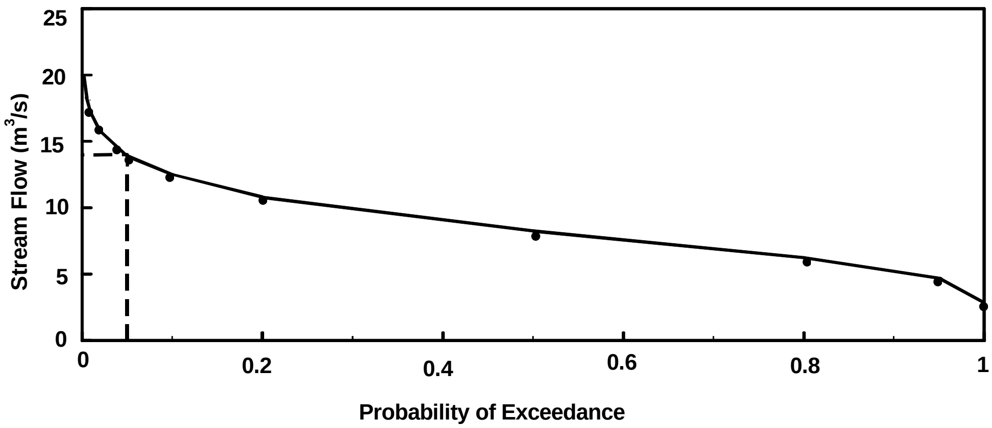
From available streamflow data, construct a data series consisting of the minimum flow, Q, observed during the period from May to September for each year of record as shown on the graphs above.

Analyze the data to determine an acceptable design low flow. The Consolidated Frequency Analysis (CFA) program was used to establish the probability function shown below.

From the graph, there is a 5% probability that the low flow during the construction period will exceed 14 m^3/s. Therefore, a low flow of 14 m^3/s would be reasonable. 95% of the time the low flow is below 14 m^3/s - 5% of the time the low flow is above 14 m^3/s.

The period of low flows is the period of time where the flows are at most 14m^3/s.

Gumbel Probability Distribution



Determine the total duration, D, for each year of record that the observed low flow was less than 14 m^3/s.

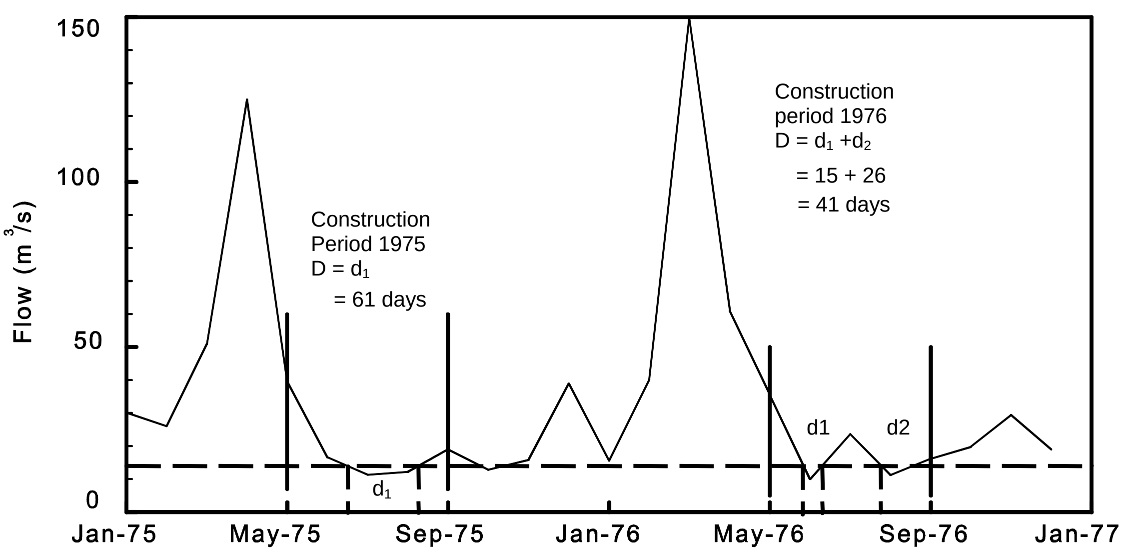
For example, as shown on the graph below,

for 1975: D_{75} $= d_1$
$= 61$ days

for 1976 D_{76} $= d_1 + d_2$
$= 15 + 26 = 41$ days

The data series consists of the total duration for each year $\{D_{15}, D_{16}, D_{17}, \ldots D_{84}, D_{85}\}$.

Daily Streamflow Data



Fishway Design Flow Estimation

The method recognizes that fish passage design takes into consideration the estimation of stream flows that will sustain the migration of fish (Katapodis, 1992). Further, it accounts for delays in fish migration.

Two assumptions are used in this method of simulating the fishway design flow as follows:

- delays of 3-days in migration; and
- delays of more than 3-days with a 10-year frequency.

The flow corresponding to the above criteria is calculated as follows from daily flow records:

- determine the 3-day delay discharge, Q_{3d}, for each year.

  Q_{3d} is the largest discharge which is equalled or exceeded in three consecutive days over the fish migration period during a particular year. This is a repetitive process that involves setting the initial Q_{3d} equal to the lowest discharge value from the first three daily discharge values from the migration period. The lowest discharge is next compared to the lowest discharge for the next 3-day period (second, third and fourth days) and the larger of the two becomes the new Q_{3d} value. This process of comparing values for 3 consecutive days is repeated for the entire migration period.

- The Q_{3d} values for each year of available data are then arranged (ranked) in descending order of magnitude. The return period (T_r) is then calculated for each value as:

  $$T_r = (n + 1) / m \qquad (8.30)$$

  where:
  n = number of years of record
  m = rank of recorded flows

- plot T_r versus Q_{3d} on log-log paper;

- the resulting line is a return period function (curve) that is used to estimate Q_{3d} (3 day delay discharge) corresponding to a 10-year return period.

If the daily flow records are not available for the stream of interest, the record of another hydrologically similar stream may be used. Flows may be extrapolated from one stream to the other based on similar climatic and basin physiographic features.

Example 8.17: Determination of Fishway Design Flows

Required

From historical records, determine the 1:10 year frequency, 3-day delay fish flow during spring migration in Coldwater Creek, at Coldwater. Spring migration occurs from May 1 to June 30.

Solution

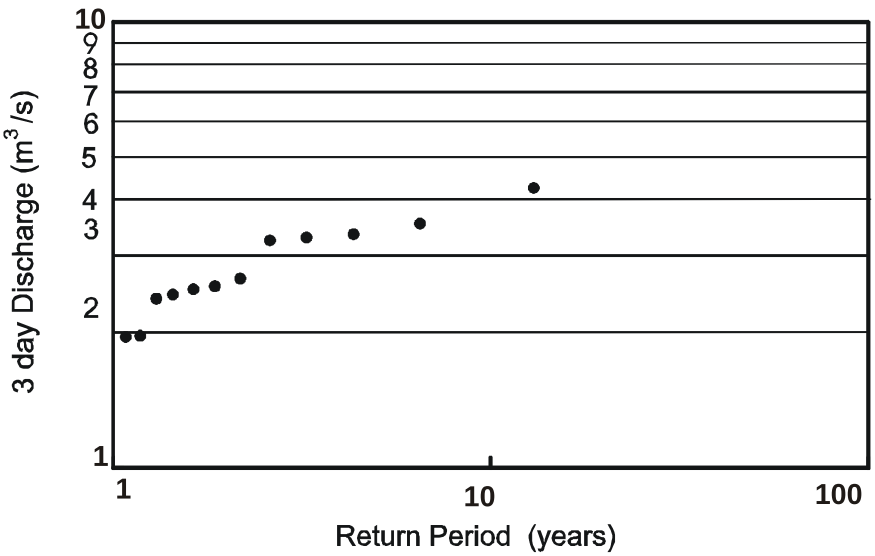
- From stream gauge records, determine the maximum discharge (Q_{3d}) that is equalled or exceeded in three consecutive days during the migration period for each year of record as shown below for 1991. This can be done by setting Q_{3dn} equal to the smallest Q_{max} for each consecutive three day period. Q_{3d} will be the largest Q_{3dn}

- Plot Q_{3d} versus return frequency (T_r) on a log-log graph where:

  $T_r = (n+1) / m$ (8.30)

  where:
  n = number of recorded years
  m = rank of recorded flows (highest = 1)

- Fit the data with a straight line to estimate the desired design flow



- The 10-year return frequency, 3-day delay fish passage flow, Q_{3d}, is 4.2 m^3/s

Hydraulic Principles of Drainage Systems Design

This section provides the basic hydraulic principles of open channel flow. Flow in streams, channels and closed systems, where the water flows under the force of gravity are classified as open channel flow.

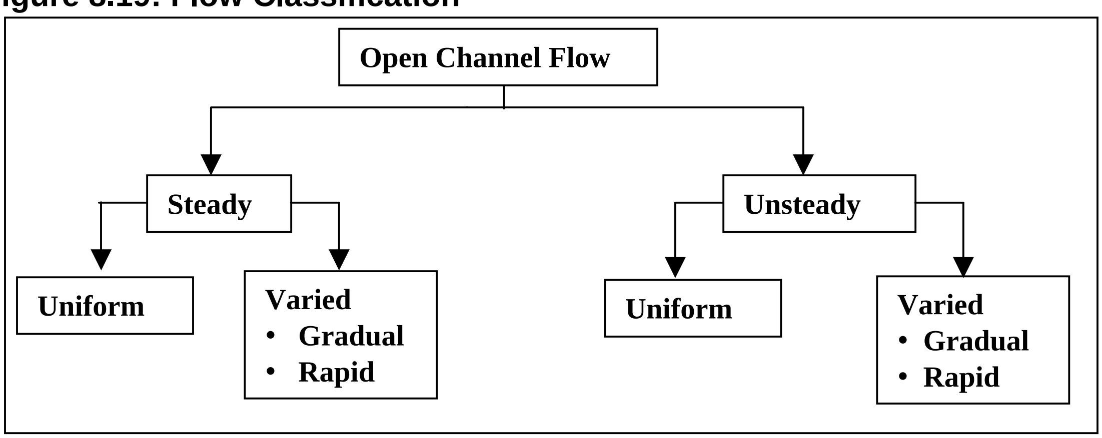
Flow Classification For Open Channel Flow

As shown in Figure 8.19, flow in open channels can be classified as: steady or unsteady, and uniform or nonuniform (gradually or rapidly varied). It can also be classified as tranquil, rapid or critical. The following is a description of these flow classifications:

- steady flow is where depth and velocity do not vary with time at a particular location (cross section);
- unsteady flow is where depth and velocity change with time at a particular location (cross section);
- uniform flow where depth and velocity do not change along a channel segment, between two cross sections of a stream;
- varied flow (gradual or rapid) where depth and velocity vary within a channel segment.
- tranquil, rapid or critical depends on the Froude number, F_r. If the F_r >1 the flow is tranquil, if F_r < 1 the flow is rapid, and if F_r = 1 the flow is critical. A more detailed discussion of this classification is covered in the section on critical depth, in this chapter.

Natural stream flow is usually unsteady gradually varied flow. However, analyzing this type of flow condition is complex and difficult to perform. Therefore, most designs are based on assuming that steady uniform or gradually varied flow conditions.

Figure 8.19: Flow Classification



Continuity Equation

The continuity principle states that discharge is constant at any cross section within a reach under uniform and steady flow conditions.

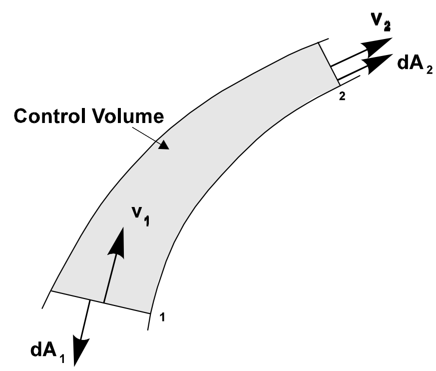
The principle of continuity of flows, gives a relationship between average velocity, depth and discharge. The flow at any cross section can be defined by multiplying the average velocity by the flow area:

$$Q_1 = A_1 * V_1 = A_2 * V_2 = Q_2 \tag{8.48}$$

where:

Q = Flow rate, m^3/s
A = Flow area, m^2
V = Flow velocity, m/s

Subscripts 1 and 2 in Equation 8.48 denote cross sections 1 and 2 as illustrated in Figure 8.20.



The principle of continuity of flow in combination with the energy and/or momentum equations is used to calculate many hydraulic variables used in the design of drainage systems, such as hydraulic jumps or backwater profiles.

Energy Equation

The principle of conservation of energy can be applied to determine flow depth and velocity for the uniform and steady, gradually varied flow condition.

The total energy at a specific point and at a specific cross section and time can be expressed as follows:

$$H = \frac{\alpha * V^2}{2g} + y * \cos\phi + z \qquad (8.49)$$

where:

- H = total energy head above datum, m
- V = flow velocity, m/s
- g = gravitational acceleration, m/s^2
- y = flow depth, m
- ϕ = longitudinal channel slope, degrees
- z = elevation of invert above datum, m
- α = energy coefficient

Bernoulli used this relationship to compare the total energy of flow between two cross sections and reasoned that the energy loss (differences in energy) is dissipated in overcoming the frictional resistance of the flow. Head loss is expressed as:

$$h_f = (y_1 + V_1^2/2g) - (y_2 + V_2^2/2g) + (z_1 - z_2) \qquad (8.50)$$

where:

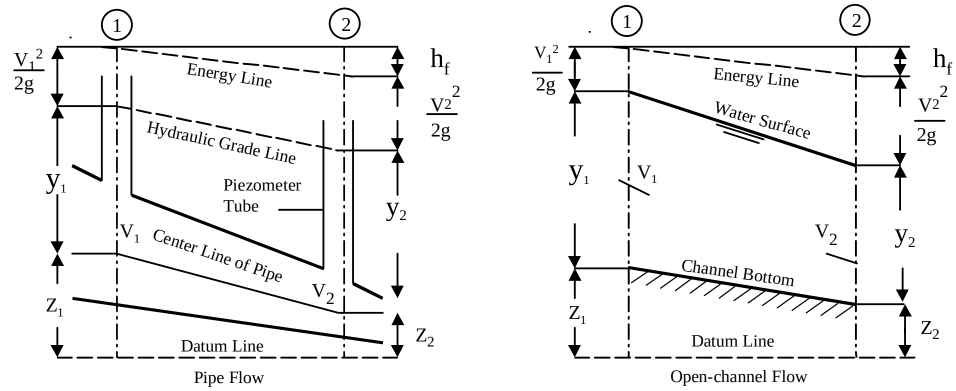
- h_f = head loss, m
- y = flow depth, m
- V = flow velocity, m/s
- g = gravitational acceleration, m/s^2
- z = elevation of invert above datum, m

Subscripts 1 and 2 represent cross sections in a reach.

The energy loss also includes expansion and contraction losses associated with exit and entry transitions and curvature of flow in a drainage system. These different types of losses are further discussed in section on energy losses.

The terms in Equation 8.50 are shown graphically in Figure 8.21. Note that for channels with small slopes and uniform flow, the energy grade line is considered to be parallel to the water surface profile (hydraulic grade line) and longitudinal bed slope.

Figure 8.21: Definition Sketch for Bernoulli Equation



Example 8.18: Application of Bernoulli Equation to Pipe Flow.

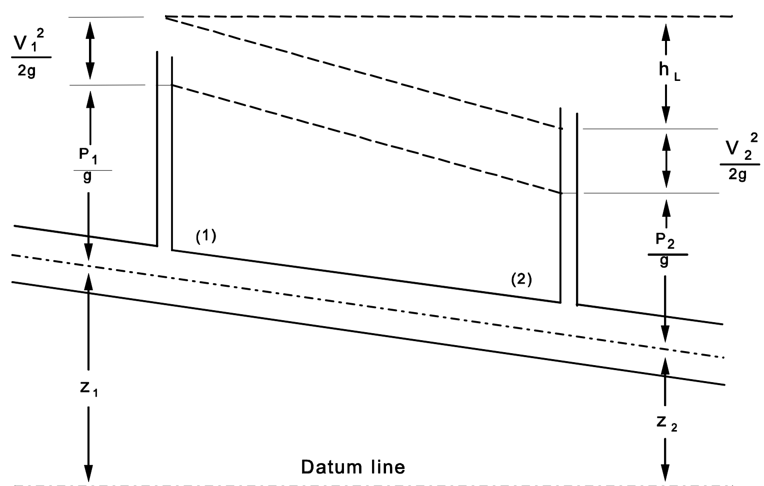
Required

Determine the flow velocity and discharge in a storm sewer pipe conveying water between two inlets.

Given

The figure below shows a typical elevation of a storm sewer pipe conveying rainfall runoff. The pipe between points (1) and (2) is 100 m long with a diameter of 300 mm. The pressure head drop (P/γ) (where γ is the specific weight of water) between points (1) and (2) is 5.0 m while the drop in elevation is 2.5 m. The Manning's roughness coefficient, n, is 0.012.

Pipe Profile and Energy Grade Line



Solution

The total energy heads at points (1) and (2) are given by Equation 8.49 rearranged.

$$V_1^2/2g + P_1/\gamma + z_1 = V_2^2/2g + P_2/\gamma + z_2 + h_l \qquad (8.51)$$

The term in Equation 8.49 for flow depth, y (m), is substituted by the quotient P/γ, where P is water pressure (N/m^2) and γ is the unit weight of water (9810 N/m^3). The resulting units are length, m.

The following continuity equation must be satisfied.

$$Q = A_1 * V_1 = A_2 * V_2 \qquad (8.48)$$

Since cross section area $A_1 = A_2$ for the sewer pipe, therefore flow velocity, $V_1 = V_2$ and the velocity head, $V_1^2/2g = V_2^2/2g$. Therefore, these terms cancel out.

Elevation change, $(z_1 - z_2)$, = 2.5 m
Pressure head change, $(P_1/\gamma - P_2/\gamma)$, = 5.0 m

Therefore, the total energy (head) lost between points (1) and (2) = 7.5 m.

This is equal to h_l, the head lost in the pipe flow through friction.

The slope, S, of the energy gradeline:

$$\begin{aligned} S &= h_l / L \\ &= 7.5 \text{ m} / 100 \text{ m} \\ &= 0.075 \text{ m/m} \end{aligned} \qquad (8.52)$$

where:
L = Length of the pipe between points 1 and 2

The slope of the energy gradeline, S, can be used to determine flow velocity through the pipe using Manning's equation as follows:

$$\begin{aligned} V &= (1/n) * R^{0.667} * S^{0.5} \\ &= (1/0.012) * \left(\frac{0.3}{2*2}\right)^{0.667} * (0.075)^{0.5} \\ &= 4.05 \text{ m/s} \end{aligned} \qquad (8.66)$$

$$Q = A * V = 0.286 \text{ m}^3/\text{s} \qquad (8.48)$$

Specific Energy

Specific energy (E) is defined as the energy per unit weight relative to the bottom of the channel for a steady, uniform flow. It is a convenient quantity to use in assessing the effect of changes in channel bottom or channel width on the water surface.

E = depth of flow + velocity head

$$E = y + V^2 / (2\ g) = y + ((Q/A)^2 / (2\ g)) \tag{8.53}$$

For a rectangular channel cross section, $A = b * y$. Accordingly,

$$E = y + (1/(2\ g))\ (Q/(b*y))^2 = y + (1/(2\ g))\ (q/y^2) \tag{8.54}$$

where:

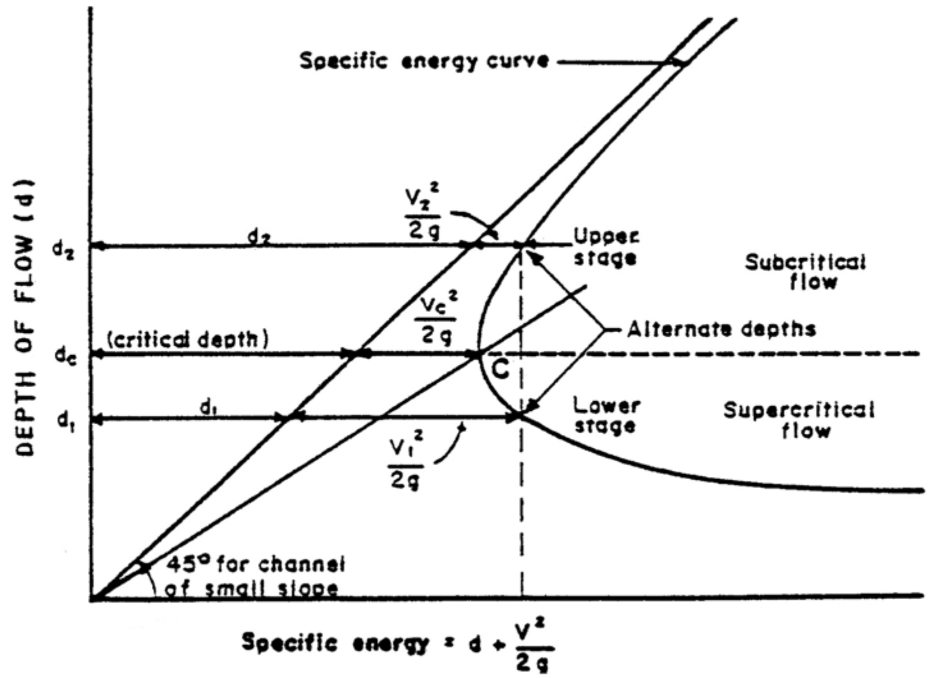
q = flow per unit width of channel

Figure 8.22 illustrates a plot of this relationship, for E versus y. Since this relationship is parabolic (i.e. $E= f\ (y^2)$), there are two values of y corresponding to each value of E. Where E is a minimum, only one value of y exists. This depth is referred to as the critical depth. For values of E greater than the minimum value, the depth of flow greater than the critical depth is referred to as the subcritical flow depth, and the depth less than the critical depth is referred to as the supercritical flow depth. Conditions in a channel may change (e.g. change in slope) such that a supercritical flow condition changes to a subcritical flow condition. In other words, the water depth goes from less than critical depth to greater than critical depth. This sudden change in water depth is what is referred to as a hydraulic jump. At a hydraulic jump strong turbulence occurs, resulting in energy loss. Therefore, for design purposes it is important to analyze the hydraulic jump to assess it's location, length and the water depth upstream and downstream to provide the necessary erosion protection and freeboard.

Notes:

- Specific energy is different from total energy in that it is measures relative to the channel bottom and not relative to a fixed datum.
- The specific energy equation is not suitable for calculation of the depth across a hydraulic jump since energy losses occur at the jump which can not be quantified.
- Since the specific energy is calculated relative to the channel bottom, any changes to the channel bottom, such as a rise or a depression, results in a change in the value of the specific energy and hence a change in the corresponding depth of flow. Consequently, a rise in the channel bed will result in a decrease in specific energy and a decrease in depth, y. This will appear as a depression in the water surface. Similarly, a depression in the channel bottom will result in a rise in the water surface.
- A change to the channel width results in a change in the velocity associated with the narrowing of the channel, hence a change in the velocity head $V^2/2g$. This results in a different curve for each value of V. However, as long as there are no changes to the channel bed, the value of the specific energy should remain constant.

Figure 8.22: Depth Versus Specific Energy Relationship



(MTO, 1992)

Critical Depth

The critical depth is an important hydraulic parameter that is an indicator of the type of flow and the need for specific design considerations in open channel design. For this purpose it is being discussed in this section. As mentioned above, critical depth is the flow depth (for a given flow rate and channel dimensions) which corresponds to the minimum specific energy.

A numerical approach to calculate actual critical depth is to use the Froude number as an index. The Froude number is a simple and convenient way of classifying flow for purposes of channel design, etc. The Froude number is the ratio of inertia forces to gravitational forces and is expressed as follows:

$$F_r = V / (g * y_m)^{0.5} \tag{8.55}$$

where:

F_r = Froude number
V = mean flow velocity, m/s
g = acceleration due to gravity, m/s^2
y_m = hydraulic mean depth, m
= flow area/ top width of flow

Froude number is used as follows to classify the state of the flow:

$F_r > 1$ supercritical flow	steep slope	fast flow, velocity dominant
$F_r = 1$ critical flow	critical slope	
$F_r < 1$ subcritical flow	mild slope	slow flow, gravity dominant

Further discussion on critical depth is presented in the sections on Specific Energy and Momentum Equation.

Energy Losses

In a drainage conveyance system, energy losses are always occurring as the flow moves through the system. The type of losses vary. Losses may be caused by friction, flow transitions, bends and turbulence. The sum of these losses is referred to as total head loss.

In hydraulic analysis, these losses are estimated and are expressed as a function of the velocity head, $V^2/2g$ (m). That is,

$$h_l = K * (V^2/2g) \quad (8.56)$$

where:

K = Loss coefficient

Friction Losses

Friction losses occur due to the shear force between the flow and the channel or pipe surface. Friction losses oppose the motion of flow. Manning's equation can be applied to provide a reasonable estimate for friction losses as follows:

$$V = (1/n) R^{2/3} * S^{1/2} \quad (8.66)$$

Substituting (Q / A) for V and solving for longitudinal slope, S:

$$S = [Q * n / (A * R^{2/3})]^2$$

By definition, for a prismatic channel:

$$S = h_f / L \quad (8.52)$$

where:

h_f = head loss due to friction, m
L = length, m

Combining and rearranging the above two equations, head loss can be expressed as a function of velocity head, $V^2/2g$ (m), as follows:

$$h_f = \frac{19.6 * n^2 * L}{R^{4/3}} * \frac{V^2}{2g} \tag{8.57}$$

where:

h_f = head loss due to friction, m
n = Manning's roughness coefficient
L = Length of pipe or channel, m
R = Hydraulic radius, m
V = flow velocity, m/s

Bend Losses

Losses associated with flows around bends are expressed in terms of velocity head as follows:

$$h_b = K_{bd} * V^2/2g \tag{8.56}$$

where:

h_b = head loss due to bend, m
K_{bd} = bend coefficient

The bend coefficient is calculated as follows:

$$K_{bd} = 0.1 * \Sigma\Delta \tag{8.58}$$

where:

K_{bd} = bend coefficient
$\Sigma\Delta$ = sum of deflection angles, degrees

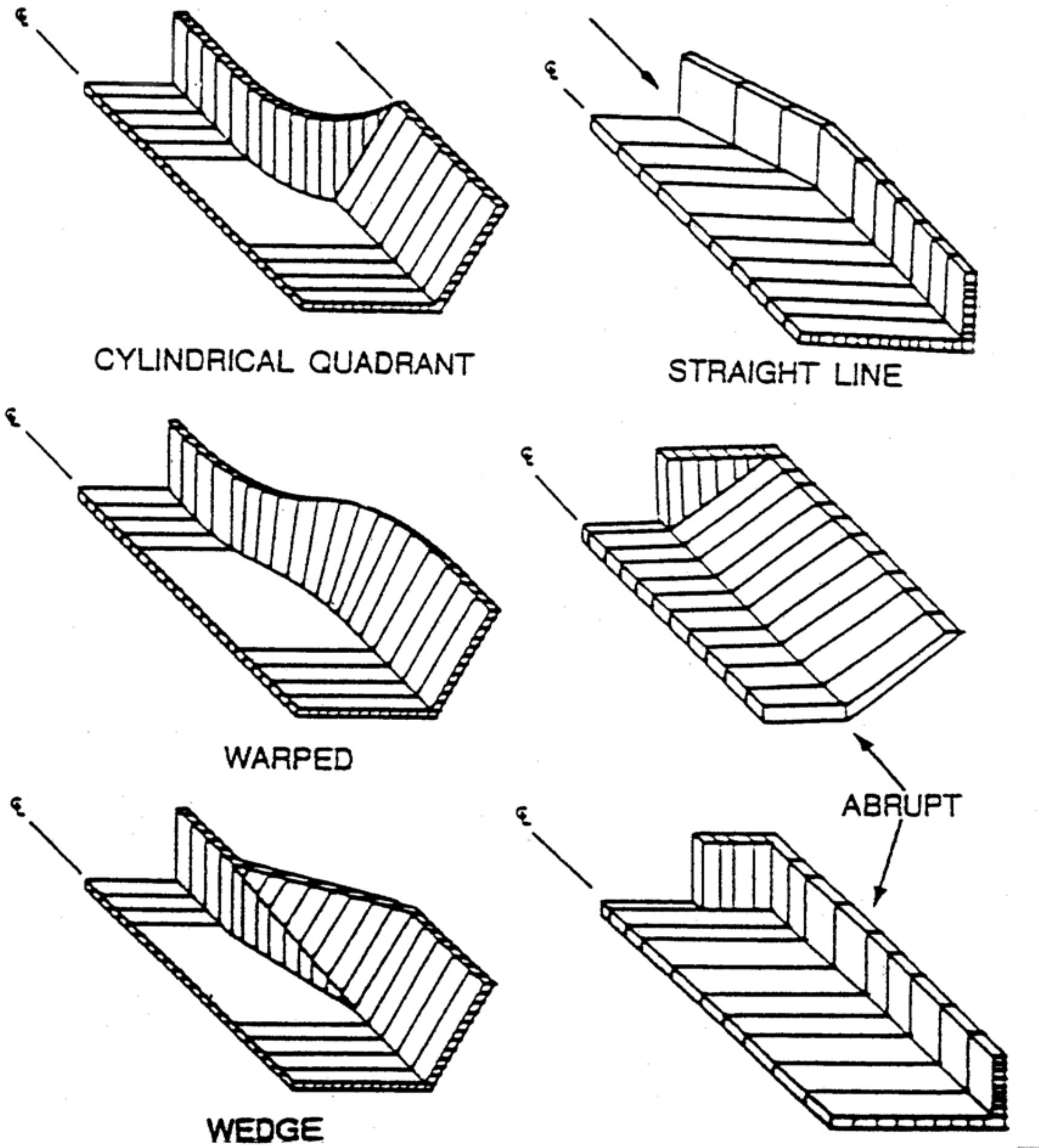
Transition Losses

Various types of transitions are shown in Figure 8.23. Losses associated with the various transitions are further discussed in Chapter 5.

Entrance and Exit Losses

Entrance and exit transition head losses are important considerations in calculating head loss at manholes, culverts, bridges and storm inlets and outlets. Entrance and exit losses are expressed in terms of the velocity head, multiplied by a K_e coefficient, similarly to other types of minor losses. K coefficients for bridges and culverts are given in Design Chart 2.07 and for storm sewer drainage systems are given in Design Chart 2.08. Detailed discussions on transition losses through bridges and culverts are discussed in Chapter 5.

Figure 8.23: Types of Channel Transitions



(MTO, 1992)

Momentum Equation

In situations where the flow is not uniform or gradually varied and flow velocity is significantly changing such as rapidly varied flow, the momentum equation is more applicable than the energy equation. The energy equation is not applicable where transitions between supercritical and subcritical flow occur, such as hydraulic jumps or at locations where flow is affected by obstacles, such as submerged weirs.

The momentum equation is derived from Newton's second law, as given in the following equation:

$$\Sigma F = M * (\Delta V/\Delta t) = M * a \tag{8.59}$$

where:

ΣF = total external force, N
M = mass, kg
ΔV = change in velocity, m/s
t = time, s
a = acceleration, m/s^2

The velocity and acceleration terms in Equation 8.59 are vector quantities. That is, they should all be in the same direction.

Therefore, for forces in the x-direction, Equation 8.59 can be written as:

$$\Sigma F_x = M * (\Delta V_x/\Delta t) = M * a_x$$

For steady flow between two cross sections (1 and 2) of an open channel, the momentum equation becomes:

$$\Sigma F = (ß_2 * \rho * Q * V_2) - (ß_1 * \rho * Q * V_1) = \rho * Q * (ß_2 * V_2 - ß_1 * V_1) \tag{8.60}$$

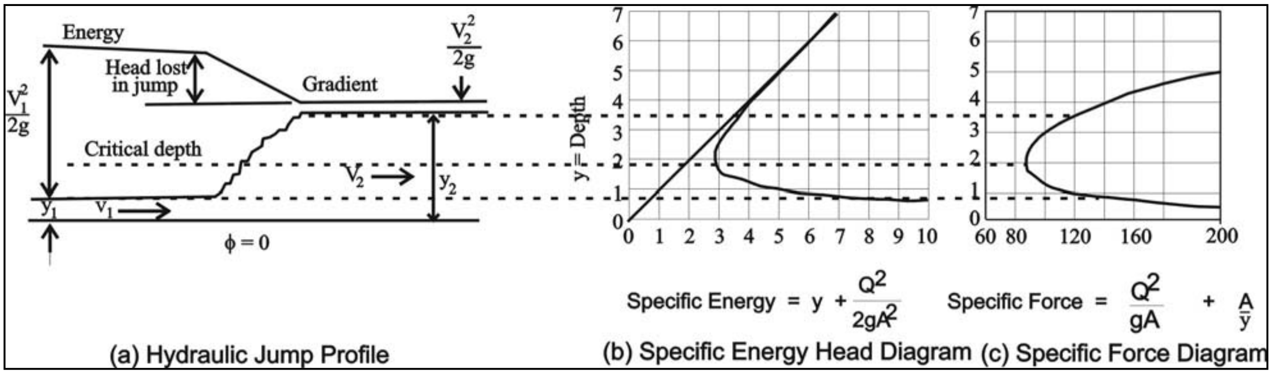
where:

ΣF = summation of all external forces in the direction of the flow, N
ß = momentum correction factor
ρ = density of water, kg/m^3
V = flow velocity, m/s
Q = flow rate, m^3/s

Subscripts 1 and 2 denote sections 1 and 2 as shown in Figure 8.24.

Equation 8.60 can be applied to determine the force exerted by a flow on drainage elements such as sluice gates, bottoms of drop structures, dissipator blocks in stilling basins, pipes bends, maintenance holes or baffle chutes.

Figure 8.24: Momentum Principle



Specific Force

Typical forces acting on an elemental volume of water include: pressure, gravity and friction. The specific force, F_s, at a section of a channel is defined by:

$$F_s = \frac{Q^2 * \beta}{g * A} + \beta' * y * \cos \phi * A \qquad (8.61)$$

where:

- F_s = specific force, m^3
- Q = flow rate, m^3/s
- ß = momentum correction factor
- g = gravitational acceleration, m/s^2
- A = flow area, m^2
- β' = pressure correction factor
- y = distance of water surface to centroid of flow, m
- ϕ = channel slope

The first term is the momentum flow per unit weight of water and the second term is pressure force per unit weight of water:

In many applications ß = ß'= 1 and cos ϕ =1 (horizontal bed), thus:

$$F_s = \frac{Q^2}{g * A} + y * A \qquad (8.62)$$

Figure 8.24 shows F_s as a function of depth. If the friction force is negligible then specific force is conserved between two points. Thus between sections 1 and 2:

$F_{s1} = F_{s2}$

and Equation 8.62 can also be written as:

$$\frac{Q^2}{g * A_1} + y_1 * A_1 = \frac{Q^2}{g * A_2} + y_2 * A_2 \qquad (8.62)$$

Subscripts 1 and 2 denote sections 1 and 2.

In the above equations, y_1 and y_2 are the distances from the water surface to the centroid of flow at the location of the alternate depths y_1 and y_2 as shown in Figure 8.24.

In applying the momentum equation, usually y_1 is the supercritical shallower depth and y_2 is the subcritical greater depth. If y_1 and Q are known, then y_2 can be computed.

By plotting the depths of flow against the specific force for a given channel, a specific force curve is obtained as shown in Figure 8.24. As shown, for a given value of specific force, there are two possible depths. The two depths correspond to the initial and sequent depths of a hydraulic jump. The initial depth, y_1, corresponds to the supercritical depth, and the sequent depth, y_2, corresponds to the subcritical depth. At the point of minimum specific energy, there is one depth ($y_1 = y_2 = y_c$) corresponding to the critical depth.

Hydraulic Jump

Hydraulic jumps are formed at the transition point where the flow regime changes from supercritical to subcritical flow, for example where a steep sloped channel transitions to a mild slope channel. Changes in flow width, bed profile and channel roughness are also factors that may create jumps. Figure 8.26 identifies the energy and momentum components of a hydraulic jump. The momentum equation is used to analyze a jump as follows:

Hydraulic Principle

$$y_1 * A_1 + \frac{Q}{(g*A_1)} = y_2 * A_2 + \frac{Q}{(g*A_2)} \qquad (8.62)$$

where:

- Q = flow rate, m^3/s
- g = acceleration due to gravity, m/s^2
- A = flow area, m^2
- y = distance of water surface to centroid of flow, m

Subscripts 1 and 2 denote cross sections 1 and 2.

For a rectangular channel, this equation can be rewritten in terms of the upstream and downstream flow depths and Froude number, F_r:

$$y_2 / y_1 = 0.5 * [(1 + 8 * F_{r1}^{2})^{0.5} - 1] \qquad (8.63)$$

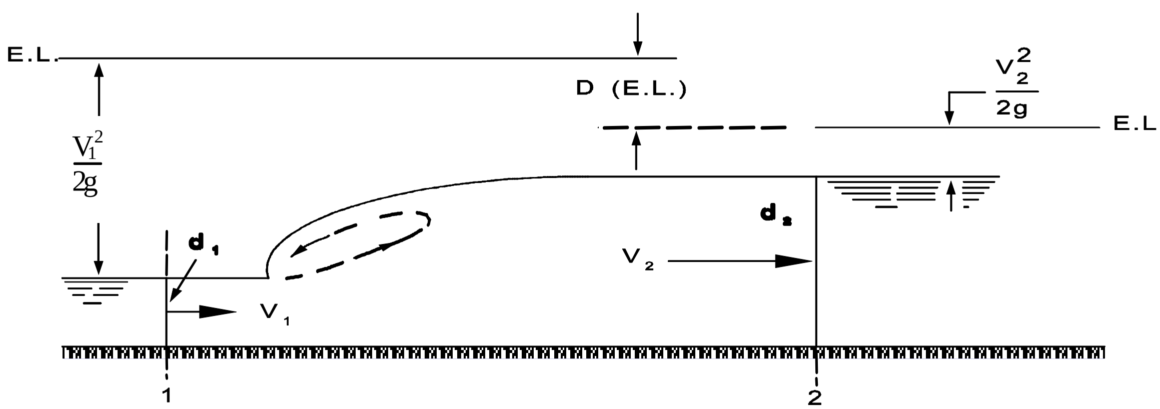
where:

F_{r1} = Froude number of the incoming supercritical flow, ($F_r > 1$);
y_1 = upstream flow depth;
y_2 = downstream flow depth.

There are many practical applications for hydraulic jumps. In highway drainage designs, they are used to dissipate energy associated with decreases in elevation.

A typical design problem would be to determine the length and position of the hydraulic jump and provide adequate protection to prevent scour downstream.

Figure 8.25: Principles of Hydraulic Jump

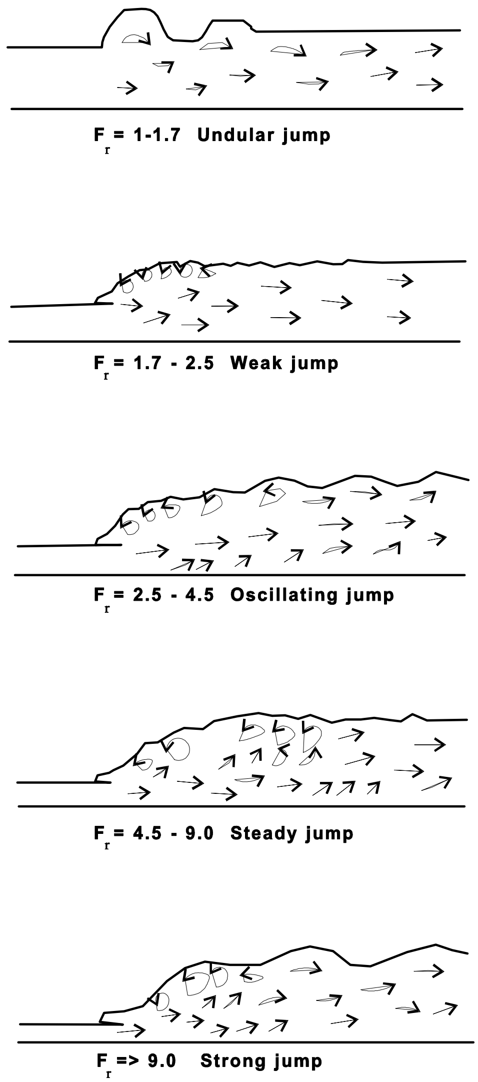


(MTO, 1992)

Types of Jumps

Jumps are classified according to the range of the Froude number. This system of classification is illustrated in Figure 8.26.

Figure 8.26: Types of Hydraulic Jump



(Chow, 1959)

Energy Loss in a Hydraulic Jump

Energy loss in a hydraulic jump can be determined from the following relationship:

$$E_1 = E_2 + \Delta E \tag{8.64}$$

where:

E_1 = Energy before the jump, m = $V_1^2/(2 * g) + y_1$
E_2 = Energy after the jump, m = $V_2^2/(2 * g) + y_2$
ΔE = change in energy, m = Energy loss in the jump
$y_{1,2}$ = flow depth upstream and downstream, m

For a rectangular channel, the energy loss across a jump can be determined directly from the following relationship:

$$\Delta E = E_1 - E_2 = \frac{(y_2 - y_1)^3}{4 * y_1 * y_2} \tag{8.65}$$

Jump Location

The hydraulic jump is located where the specific force of the flow in the upstream channel is equal to the specific force of the flow in the downstream channel. The jump can occur either in steep or the mild sloped sections.

The position of the jump is where y_1, y_2 and F_r satisfy Equation 8.63.

Theoretically, the jump is located where supercritical flow from the upstream direction intersects subcritical flow (backwater effects) from the downstream direction.

Jump Length

It is important to reasonably estimate the length of a jump so that adequate channel surface protection may be specified. In a stilling basin, the designed length depends on the length on the jump, so that the jump can be confined within the apron to reduce the likelihood of scour downstream.

The length of a jump is often estimated as a factor multiplied by the sequent depth. Experimentally, it has been shown to be related to the Froude number. Jump length may be estimated, using Design Chart 2.20.

Height of Jump

This is the difference between the depths after and before the jump ($y_2 - y_1$).

Example 8.19: Hydraulic Jump

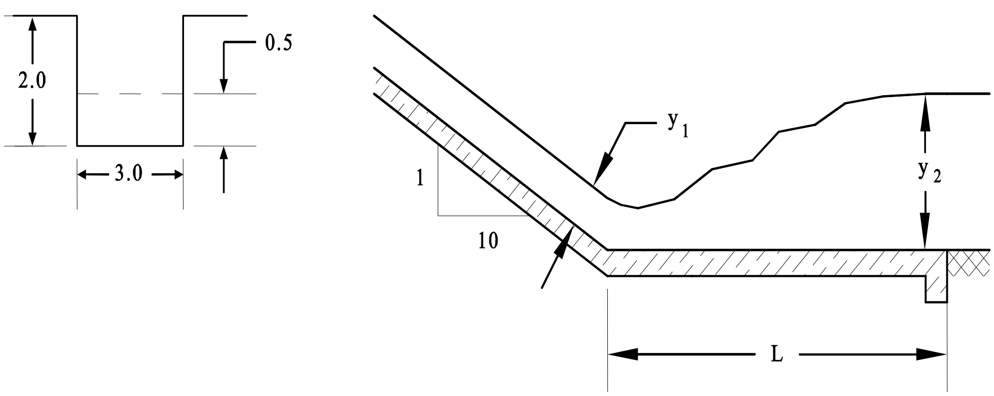
Required

A diversion chute conveys relief flow from a main channel into a diversion channel in order to reduce the size of a proposed highway crossing waterway opening. Determine the characteristics of the hydraulic jump at the bottom of a chute.

Given

A typical cross section of the chute and jump is shown in the figure below. The diversion chute is concrete and rectangular and has a constant slope of 10%. The depth of flow in the drop chute is less than 1.5 m.

Assume chute bottom width, b	= 3.0 m
Manning's coefficient for concrete, n	= 0.012
Discharge, Q	= 20 m^3/s
Chute slope, S	= 0.10 m/m



Solution

- Calculate Sequent depth y_2:

Applying Manning equation,

$$Q = A * V = \frac{A * R^{2/3} * S^{1/2}}{n} \quad (8.48)$$

By trial and error, for a depth of flow, $y_1 = 0.50$ m

$A = 0.5 * 3.0 = 1.5\ m^2$, $P = 3.0 + (2 * 0.5) = 4.0$ m, $R = 1.5/4.0 = 0.375$

$$Q = A * V = \frac{1.50 * 0.375^{2/3} * 0.10^{1/2}}{0.012} = 20.56 \text{ m}^3/\text{s} \quad (8.48)$$

Q is approximately 20 m^3/s, therefore, iterations are complete. If the result of the above calculation is not close to the given discharge, Q, a revised depth, y_1, would be estimated and the calculation reiterated.

The flow velocity,

$$V = Q / A \quad (8.48)$$
$$= 20.56 / 1.5 = 13.7 \text{ m/s}$$

Froude number,

$$F_r = V_1 / (g*y_1)^{1/2} \quad (8.55)$$
$$= 13.7 / (9.81 * 0.5)^{1/2} = 6.2 \quad (F_r > 1, \text{ therefore flow is supercritical})$$

The ratio y_2/y_1 for a rectangular channel may be obtained using the following equation,

$$y_2/y_1 = 0.5 * ((1 + 8 * F_r^2)^{0.5} - 1) \quad (8.63)$$

Calculate y_2,

$$y_2 = 0.5 * ((1 + 8 * 6.2^2)^{0.5} - 1) * y_1$$
$$= 0.5 * ((1 + 8 * 6.2^2)^{0.5} - 1) * 0.5 \text{ m} = 4.14 \text{ m}$$

- Calculate energy loss:

Energy loss through the hydraulic jump is determined using the following equation:

$$\Delta E = (y_2 - y_1)^3 / 4(y_1 * y_2) \quad (8.65)$$
$$= (4.14-0.5)^3 / (4 * 0.5 * 4.14) = 5.82 \text{ m}$$

- Calculate the length of the jump:

$$F_r = 6.2$$
$$L/y_2 = 6.1 \quad \text{(Design Chart 2.20)}$$
$$L = 6.1 * y_2$$
$$= 6.1 * 4.14 \text{ m} = 25.2 \text{ m}$$

Conclusion

This jump occupies a great length and would require that channel lining protection be placed for at least this length. In this case, it may be more practical to incorporate a stilling basin design that would greatly reduce the length of the jump and associated channel protection measures.

Manning Equation

Several empirical equations have been used to estimate velocity and discharge in open channels assuming steady flow conditions. The Manning equation is widely used in design applications as it is simple to use and provides reliable estimates.

$$V = (1/n)\, R^{2/3} * S^{1/2} \qquad (8.66)$$

where:

- V = flow velocity, m/s
- n = Manning's roughness coefficient
- R = hydraulic radius, m (flow area/wetted perimeter)
- S = channel slope, m/m

The Manning's equation is appropriate for steady uniform flow conditions, such as those occurring in the mid-section of a long channel beyond the influence of flow disturbances. The channel, in the vicinity of this zone, would have a reasonably uniform slope and cross section shape.

Typical Manning's roughness coefficients are shown in Design Chart 2.01. Roughness coefficients vary according to lining material, flow depth, velocity and sediment load. However, surface roughness coefficients are usually assumed constant for design applications.

For grass lined channels, variation in roughness should be considered. Experimentally, it has been shown that Manning's roughness coefficients for vegetal linings varies with the product of the flow velocity and depth of flow, as illustrated in Design Chart 2.01.

For riprap lining, Manning's roughness coefficients can be estimated for different sizes of stones using the following equation:

$$n = 0.0152 * D_{50}^{1/6} \qquad (8.67)$$

where:

- D_{50} = 50th percentile stone size, mm

Where necessary, sensitivity analyses may be conducted to determine the impact of the roughness coefficients on flow depths, velocities and tractive forces.

The following equation can be applied to determine a composite n-value, n_c, for artificial and natural channels with various lining materials with differing n values:

$$n_c = \frac{\Sigma(A_i * R_i^{2/3} * n_i)}{A * R^{2/3}} \qquad (8.68)$$

where:

n_c = composite Manning's roughness coefficient
n_i = Manning's roughness for section i
A_i = flow area for section i, m^2
R_i = hydraulic radius for section i, m
A = flow area for total cross section, m^2
R = hydraulic radius for total cross section, m

Example 8.20: Uniform Flow in Composite Channel Using Manning Equation.

Required

Determine the flow velocity and discharge in the composite trapezoidal channel shown in the figure below.

Given

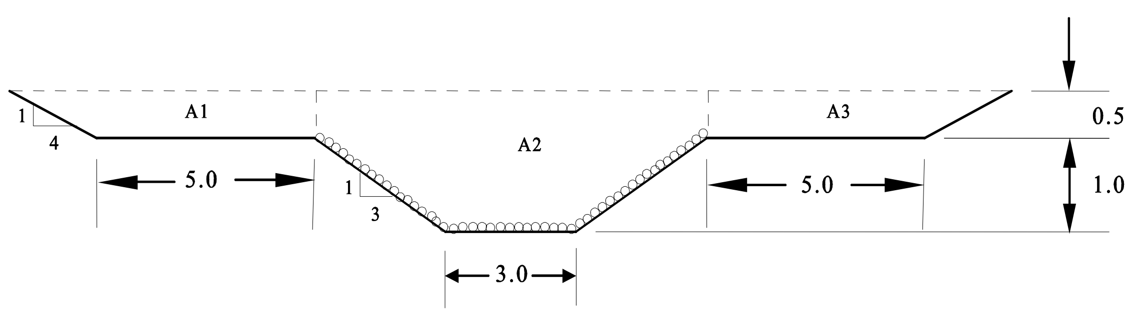
Low flow Channel:

bottom width, b_w	= 3 m
side slope, Z_l	= 3
lining material dia., D_{50}	= 150 mm
bed slope, S	= 0.005 m/m
depth, d_l	= 1.0 m

Upper channel:

bottom width, b_{wul} = b_{wur}	= 5 m
side slope, Z_u	= 4
lining material	= vegetal
bed slope, S	= 0.005 m/m
depth, d_u	= 0.5 m
total depth	= 1.5 m

Channel Cross section



Solution

- Calculate Manning's roughness coefficient, n_l, for lower channel with riprap lining as follows:

$$n_l = 0.0152 * D_{50}^{1/6} \quad (8.67)$$
$$= 0.0152 * 150^{1/6}$$
$$= 0.035$$

- Obtain Manning's roughness coefficient, n_u, for the upper channel for grassed channels and swales for the following conditions: (Design Chart 2.01)
  - fair stand, any grass, length 0.3 m
  - flow depth, d = 0.5 m
  - flow velocity, V = 0.6 m/s to 1.8 m/s

Manning's roughness coefficient, n = 0.06, for the upper channel.

- Calculate a composite roughness coefficient, n_c, for the entire channel cross section, as follows:

$$n_c = \frac{\Sigma(A_i * R_i^{2/3} * n_i)}{A * R^{2/3}} \quad (8.68)$$

- Summarize input parameters A_i, A, R_i, R, and n_i.

A_1 = 3.0 m^2; R_1 = 0.42 m;n1 = 0.06;
A_2 = 10.5 m^2; R_2 = 1.13 m;n2 = 0.035;
A_3 = 3.0 m^2; R_3 = 0.42 m;n3 = 0.06;
A = 16.5 m^2; R = 0.70 m [= A / P = 16.5 m^2 / 23.5 m]

Substitute in Equation 8.68, the composite Manning's coefficient, n_c = 0.046

- Use Manning equation calculate discharge and velocity as follows:

$$V = (1/n) R^{2/3} * S^{1/2} \quad (8.66)$$

$$V = \frac{0.70^{2/3} * 0.005^{1/2}}{0.046}$$

$$Q = \frac{A * R^{2/3} * S^{1/2}}{n}$$

$$Q = \frac{16.5 * 0.70^{2/3} * 0.005^{1/2}}{0.046}$$
$$= 20.0\ m^3/s.$$

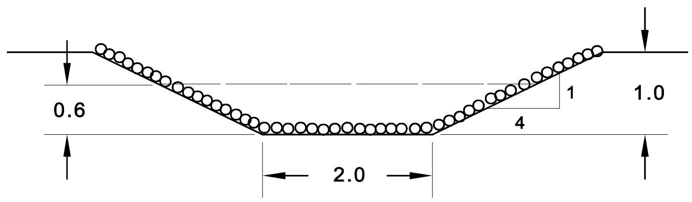
Example 8.21: Normal and Critical Flow Depths in a Uniform Flow Channel Using Manning's Equation.

Required

Determine the normal and critical flow depth in a roadside ditch conveying runoff from a 10-year storm event.

As shown in the figure below, the channel is trapezoidal and lined with riprap with D_{50} equal to 150 mm. It has a bed slope of 0.005 m/m.

Given



Bed width, b_w = 2 m
Bed slope, S = 0.005 m/m
Side slope, z = 4.0
Design flow, Q = 2.8 m^3/s

Solution

Calculate Manning's roughness coefficient, n, for a channel with riprap lining as follows:

$$n = 0.0152 * D_{50}^{1/6} \tag{8.67}$$
$$= 0.0152 * 150^{1/6}$$
$$= 0.035$$

Calculate normal depth using the Manning equation

$$Q = \frac{A * R^{2/3} * S^{1/2}}{n} \tag{8.66}$$

Solve by trial and error, start by assuming the flow depth, d, equal to 0.5 m. Calculate A, R and substitute in the Manning equation. For a trapezoidal channel:

$$A = b_w * d + z * d^2$$
$$= 2.00 \text{ m}^2$$

$$P = b_w + 2 * d * (1+z^2)^{0.5}$$
$$= 6.12 \text{ m}$$

$$R = A / P$$
$$= 2.00 \text{ m}^2 / 6.12 \text{ m} = 0.327 \text{ m}$$

$$Q = \frac{2.00 * 0.327^{2/3} * 0.005^{1/2}}{0.035} \qquad (8.66)$$
$$= 1.92 \text{ m}^3\text{/s.}$$

Reiterate with a trial flow depth, d = 0.6 m:
A = 2.64 m^2
P = 6.95 m
R = 0.380 m
Q = 2.80 m^3/s

Since the resulting Q = 2.80 m^3/s is equal to the design Q, the normal flow depth is 0.60 m.

Calculate the critical depth

Critical depth (d_c) occurs where the Froude Number, F_r = 1. Critical Depth is calculated from Equation 8.55 (refer to the section on Critical Depth further in this chapter) using F_r = 1.

$$F_r = V / (g * y_m)^{0.5} \text{ [=1 for critical flow]} \qquad (8.55)$$
$$= V / (g * A/T)^{0.5}$$

where:
A = cross section area, m^2
T = top width, m

For critical flow, this gives:

$$V / (g * A/T)^{0.5} = 1 \qquad (8.55)$$

$$V^2 / (g * A/T) = 1$$

and V = Q/A

$$\frac{Q^2 * T_c}{g * A_c^3} = 1 \qquad (8.69)$$

where:
T_c = top width at critical depth, m
A_c = area at critical depth, m^2

Reformulating to solve for Q as a function of critical depth, d_c; bottom width, b_w; and side slope ratio, z.

$$Q = d_c^{1.5} * [g * (b + z * d_c)^3 / (b + 2 * z * d_c)]^{0.5} \qquad (8.70)$$

By trial and error;
$d_c = 0.42$ m and $T_c = 5.4$ m

Critical depth = 0.42 m

Flow Measurements and Control

For calibration and verification of hydrologic and hydraulic models, flow measurements are required. Sometimes, it is possible to undertake a direct measurement like stream gauging rather than other structural methods using control devices such as weirs and orifices. This section discusses these methods.

It should be recognized that the use of control devices is not limited to measurement of flow. They also provide a useful method of hydrologic control, such as regulating the outflow from a detention pond or controlling flow in a channel.

Stream Flow Measurements

In the absence of stream flow gauging stations, stream flow measurements may be needed to calibrate and validate hydrologic or hydraulic estimates or simulations. In this way, the accuracy of hydrologic estimates are highly improved, resulting in a more economical and efficient design of drainage systems.

If appropriate, temporary gauges may be installed to collect data of stage and discharge at a location in a stream where hydrologic estimates are desired.

For stream flow measurements, hydraulic control devices such as weirs, orifices, notches, and flumes are often used. More direct and elaborate methods include measurements using:

- velocity - area integration with current meters;
- velocity - time of travel with floats;
- tracer - dilution; and
- ultrasonic.

There are also indirect methods which involve the principles of fluid dynamics, in which the continuity equation, energy equation and the momentum equation are applied to derive a single discharge equation. Flows determined by the Manning equation is a simplified example.

A standard book on hydrometry may contain greater detail of different methods of stream gauging.

Stage-Discharge Curve

Discharges at a stream crossing may be estimated from a stage-discharge relationship typically shown in Figure 8.27 that may be derived from flow records.

A stage-discharge rating curve depicts the relationship between stage and discharge at a point of a stream. The curve is established from several field measurements and can be used to interpolate flows or water levels if either is known.

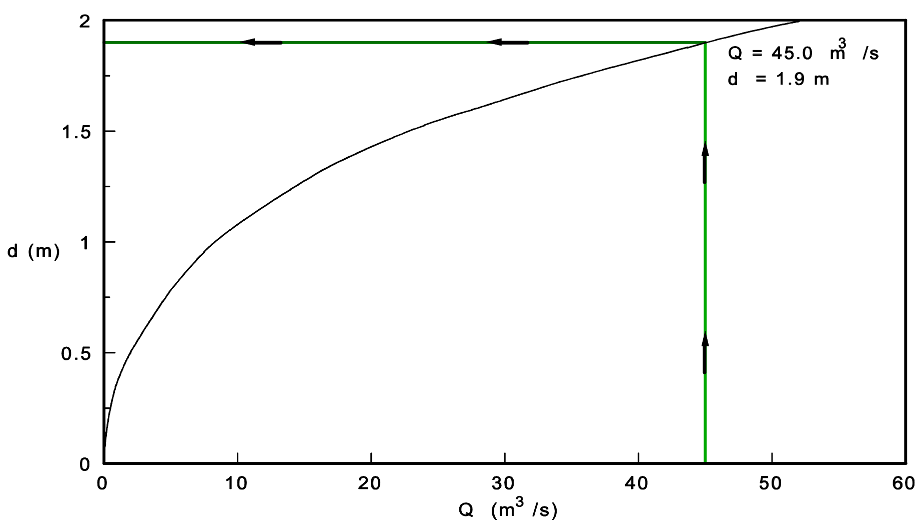
A simple stage-discharge relationship can be expressed as:

$$Q = A * y^n \tag{8.71}$$

where:

- Q = discharge, m^3/s
- A = constant
- y = gauge height measured from bottom of stream, m
- n = constant

Figure 8.27: Typical Storage-Discharge Curve

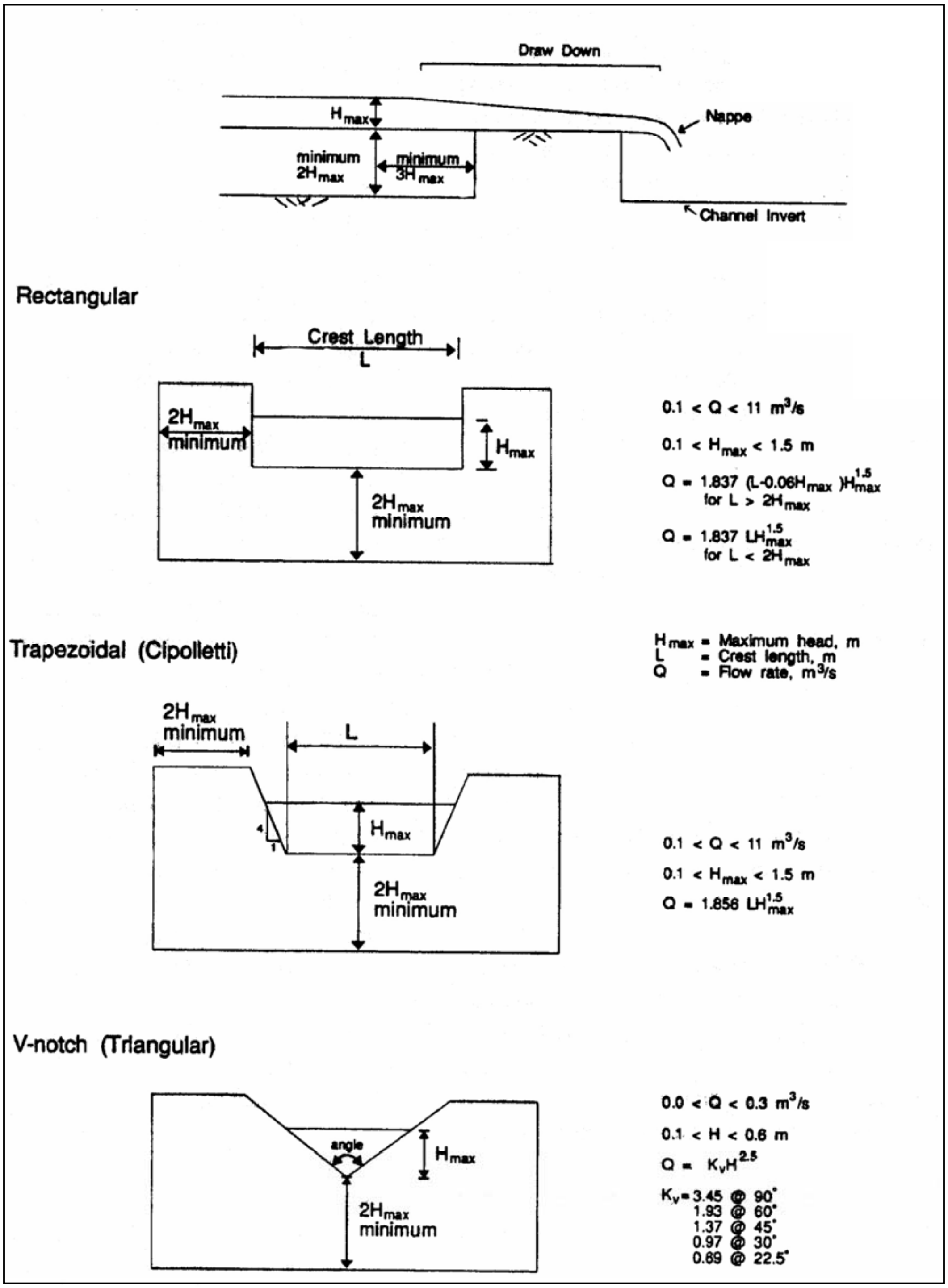


(MTO 1986)

Control Devices

Control devices discussed in this section include weirs and orifices. Figure 8.28 summaries the various types of weirs that can be used in the design of drainage systems. Each of these controls is discussed below.

Figure 8.28: Types of Weirs



(MTO, 1992)

Flow Over Weirs and Notches

A weir is a flow control device used in drainage systems to control discharges, typically in detention ponds. It can also be used as a flow measuring device by measuring the head over the weir and converting it into a discharge by knowing the head-discharge relationship. A weir may consists of a flat vertical plate, in this case it would be known as sharp-crested weir, or a solid broad section, this would be referred to as a broad-crested weir. Weirs may be classified according to their shape, rectangular, triangular, trapezoidal (Cipoletti) or parabolic. The most common geometric shapes of weir structures and their corresponding head-discharge relationships are shown in Figure 8.28 (MTO, 1992).

For the flow over a weir to follow the head-discharge relationships, shown in Figure 8.28 and described in the following sections, the downstream water level must be lower than the crest. If the downstream water level is high such that it effects the flow over the weir, a situation referred to as a flood out situations, the submergence effect should taken into account and evaluated.

Rectangular Sharp-Crested Weir

Water flowing over a sharp-crested weir under free discharge conditions is shown in Figure 8.28. Air is usually trapped between the nappe and the weir. The pressure of the trapped air is below atmospheric pressure and has the effect of increasing the discharge of the weir. This may cause damage to the downstream channel. Proper design of sharp-crested weirs should include ventilation pipes to release this pressure differential.

The discharge over a sharp-crested weirs can be estimated from Smith (1978) formula:

$$Q = 1.837\ b * h^{3/2} \qquad (8.72)$$

where:

b = crest length of weir, m
h = upstream head, m, measured vertically from weir crest to the water surface (at least 3 h distance upstream of the weir).

Two major factors that influence the discharge are the approaching flow velocity and side contractions for the flow.

If b/h > 2, the weir crest length should be reduced to account for the effects of side contractions. For such weirs, discharge can be calculated using:

$$Q = 1.837 * (b - 0.06h) * h^{3/2} \qquad (8.73)$$

The discharge from these equations should be further adjusted for the effects of submergence using Design Chart 2.47.

For cases where the weir width, b, is less than the channel width, B, Design Chart 2.42 gives the weir coefficient, adjusted for rectangular contractions for different b/B ratios.

Example 8.22: Sharp-Crested Rectangular Weir

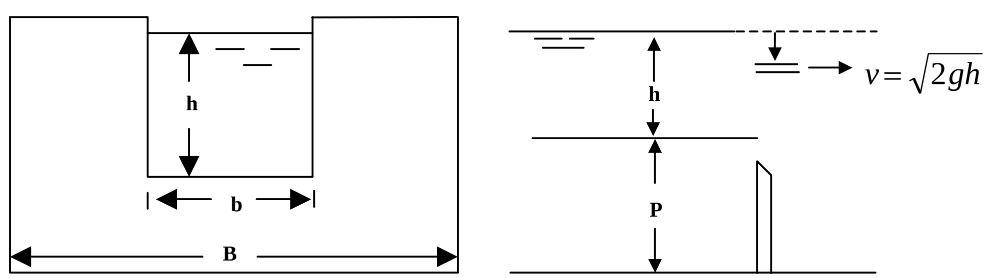
Required

The cross section and longitudinal elevation of a sharp crested weir is shown in the figure below. It is a standard, uncontracted, horizontal weir where width, L, is equal to the channel width, B.

Given

Weir crest length, b = 5.0 m
Weir height, P = 0.5 m
Upstream headwater above crest, h = 0.5 m
Downstream headwater above crest, h_s = 0.25 m

Solution



Crest length b/h > 2
Discharge Q = 1.837 * (b - 0.06h) * $h^{3/2}$ (8.73)
Apply submergence factor (Design Chart 2.47)

h_s/h = 0.5
C_s/C = 0.85

Therefore,

$$Q = (0.85)(1.837)(b - 0.06h)h^{3/2} \quad (8.73)$$
$$= 2.75 \text{ m}^3/\text{s}$$

Broad-Crested Rectangular Weir

As water flows over a broad-crested weir, the flow is accelerated and the head drops as illustrated in Figure 8.28. Smith (1979) estimated the discharge over these types of structures as:

$$Q = C_d * b * H^{3/2} \tag{8.74}$$

The weir coefficient, C_d, is generally taken as 1.705. It has been determined experimentally that the value of C_d increases with an increase in H/L, where L is the longitudinal dimension of the weir (length), as indicated in Design Chart 2.43. Adjustment should also be made to account for submergence effects using Design Chart 2.47.

Example 8.23: Broad-Crested Rectangular Weir.

Required

Determine the discharge passing over a broad-crested weir when the dimensions of the weir, and the upstream and downstream water levels are known.

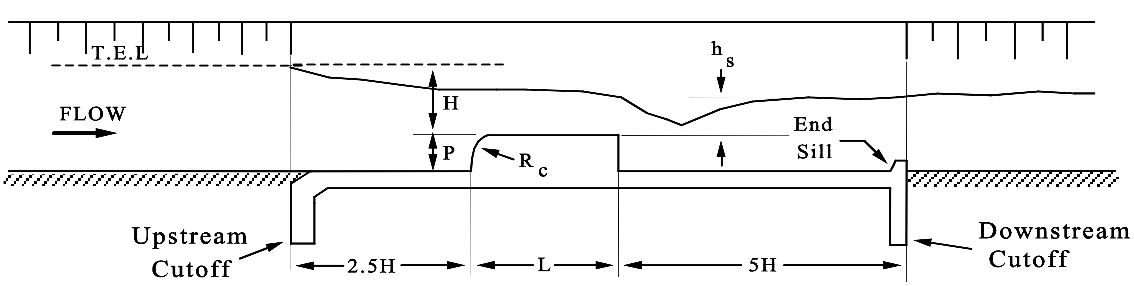
Given

A typical longitudinal elevation of the weir is shown in the figure below. The measured upstream flow depth is 1.0 m and the downstream flow depth is 0.75 m.

weir height, P = 0.5 m
weir width, b = 5.0 m
weir length, L = 2.0 m

Upstream flow depth (H + P)= 1.0 m

Therefore, flow depth over weir, H = 1.0 - 0.5 = 0.5 m



℄ Section

Solution

h_s = Downstream depth - P

h_s = 0.75 - 0.5 = 0.25 m

The general weir equation is as follows:

$$Q = C_d * b * H^{3/2} \quad (8.74)$$

Determine adjustments to the weir coefficient. (Design Chart 2.43)

H/L = 0.5/2 = 0.25
and the coefficient C = 1.63

Correct for submergence:
h_s/H = 0.25/0.5 (Design Chart 2.47)
= 0.5

For values of $h_s/H \le 0.65$, no correction of C is required, or $C_s/C = 1$, where C_s = reduced coefficient under submergence effect

$$Q = 1.63 * b * H^{3/2} \quad (8.74)$$
$$= 1.63 * 5 * 0.5^{3/2}$$
$$= 2.88 \text{ m}^3\text{/s}$$

Triangular (V-notch) Weir

Triangular weirs are suitable for low flow discharges because the head increases more rapidly as a function of flow rate on a triangular section.

Figure 8.28 shows the typical V-notched weir. Discharges can be computed using:

$$Q = C_d * (8/15) * (2g)^{0.5} * \tan(\theta/2) * h^{5/2} \quad (8.75)$$

Design Chart 2.44 gives the discharge coefficient C_d for different angles as a function of h. In cases where there is a contraction at the weir (the width at the top of the V-notch weir is less than the channel width B), Design Chart 2.45 can be used to obtain the value of C_d for the 60° and 90° V-notch weirs, and different channel widths.

For broad-crested weirs, Design Chart 2.46 gives the value of the coefficient C as a function of h/L, where L is the longitudinal dimension (length) of the weir.

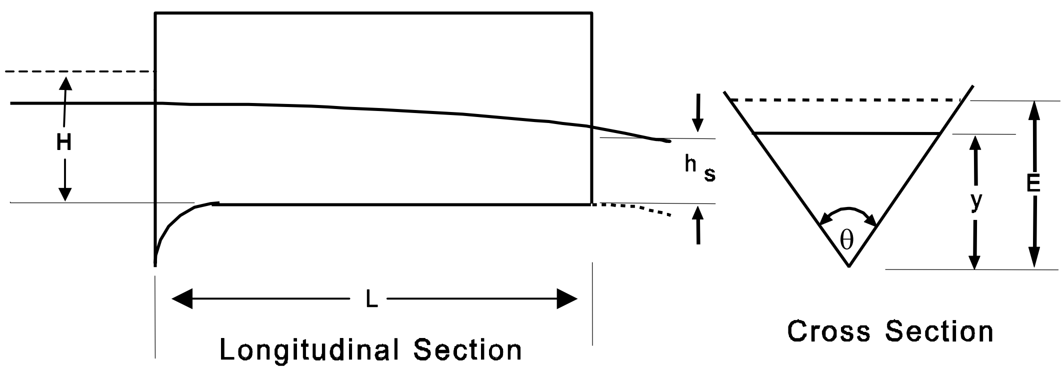
Example 8.24: Broad-Crested Triangular Weir.

Determine the discharge passing over a broad-crested triangular weir when the dimensions of the weir, and the upstream and downstream water levels are known. A typical longitudinal elevation of the weir is shown in the figure below.

Given

The cross section is triangular with θ equal to 90 degrees.
Upstream headwater above crest, H = 0.5 m
Downstream headwater above crest, h_s = 0.25 m
L = 2.0 m

Weir Cross section



Solution

The equation for discharge over a triangular broad-crested weir is given by:

$Q = C * H^{5/2} * \tan(\theta/2)$ (8.76)

Adjust for broad-crested geometry (Design Chart 2.46)
H/L = 0.25
C = 1.225

Adjust for submergence (Design Chart 2.47)
h_s/H = 0.25/0.5
= 0.5
C_s/C = 1 (no correction required)

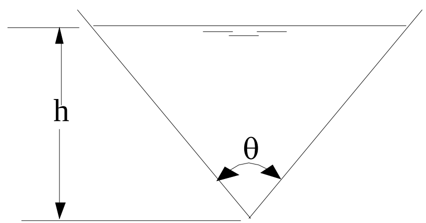
$Q = 1.225 * H^{5/2} * \tan(\theta/2)$
$= 1.225 * (0.5)^{5/2} * \tan 45^o$
$= 0.22\ m^3/s$

Example 8.25: Sharp-Crested Triangular Or V-Notch Weir

Example 8.22 is repeated for the sharp-crested V-notch weir. The cross section of the V-notch weir is shown in the figure below.

Given

Upstream headwater above crest, h = 0.5 m
Downstream headwater above crest, h_s = 0.25 m
$\theta = 90^o$



Solution

Equation for discharge for sharp crested V-notch weir is as follows :

$$Q = C_d * (8/15) * (2g)^{0.5} * \tan(2/2) * h^{5/2} \qquad (8.75)$$

Calculate the weir coefficient C_d (Design Chart 2.44)

For heads greater than 0.35 m
C_d = 0.58

Correction for submergence
h_s/H= 0.5 (Design Chart 2.47)
C_s/C = 0.98

Therefore,
C_s = 0.98 x 0.58
= 0.568

Therefore,
$Q = C_s * (8/15) * (2g)^{0.5} * \tan(1/2) * h^{5/2}$
$= 1.34 * h^{5/2}$
$= 1.34 * (0.5)^{5/2}$
$= 0.24\ m^3/s$

Flow Over Embankment

Estimates of flow over embankments (highways) are needed at many bridge and culvert locations to determine flow characteristics, such as water levels and velocities, in the event that the highway is overtopped. Flow that is accommodated in this manner is referred to as relief flow. This type of design may be appropriate for minor roads. However, for major highways, overtopping by flow may not be permitted.

Flow over an embankment may be estimated by: (Bradley J.H., FHWA, 1978)

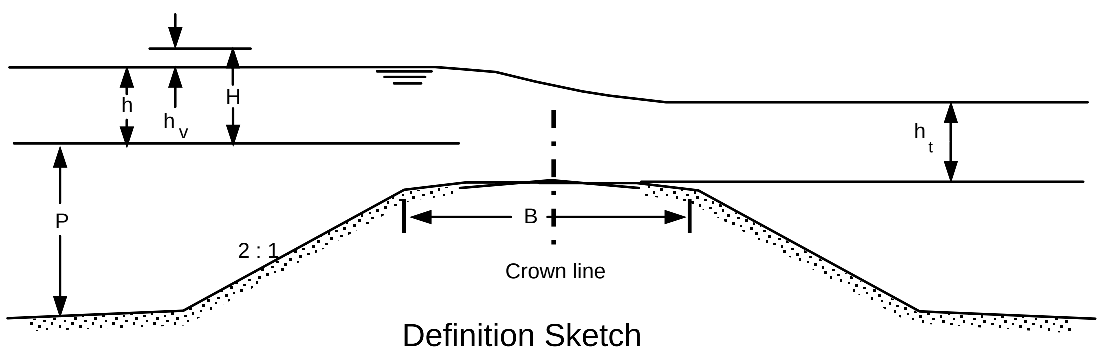
$$Q_o = 0.55 * k_t * C * L * H_o^{1.5} \qquad (8.77)$$

where:

- Q_o = total overflow, m^3/s
- k_t = submergence factor
- C = discharge coefficient
- L = length of overflow along embankment, m
- H_0 = h, if approach flow velocity, $V_1 < 2$ m/s; otherwise, $H_o = h + V_1^2/2g$, m

Figure 8.29 shows the definition sketch of an embankment type of flow. Design Chart 2.09 provides values for discharge coefficient, C, and submergence factor, k_t.

Figure 8.29: Flow Over Embankments



(MTO 1986)

Example 8.26: Flow Over Embankment

Determine the relief flow passing over the sag in a roadway embankment when the width and elevation of the embankment are known, as well as the upstream and downstream water levels.

Given

A typical cross section of flow over an embankment is shown in the figure below.

Average upstream flow depth = 1.67 m
Downstream flow depth = 0.8 m. (Tailwater depth, h_t= 0.0 m)
Overflow embankment length, L = 200 m at a maximum depth of 1.0 m
Embankment height = 1.0 m

The downstream water level will remain below the crest of the roadway.

Roadway width, b = 20.0 m
Maximum headwater depth, h = 2.0 m
Design storm, Q > 25-year event

Solution

The general weir equation for flow over a roadway embankment is as follows:

$$Q_o = 0.55 * E (k_t * C * L * H_o^{3/2}) \tag{8.77}$$

Since road sags are represented by vertical curves, the depth of flow over a road embankment varies along the overflow section. To calculate discharge, the embankment is divided into longitudinal subsections, each having a fairly uniform depth of flow. Coefficients C and k_t are determined (Design Chart 2.09). The discharge for each subsection is then calculated using the method outlined in this example. Discharges from individual subsections are summed to obtain flow over the embankment.

For simplicity, this example uses an average depth of 1.67 m for the overflow length of 200 m.

The total head H = 1.67 - 1.0 = 0.67 m

Determine weir coefficient, C (Design Chart 2.09)

for h/b = 0.04, and
H_o= h = 0.67 m
Weir coefficient, C = 3.04

Correct for submergence

h_t/H_o = 0 (Design Chart 2.47)
k_t = 1.0, Therefore, no correction for submergence is required.
Q_o = $0.55 \times 1 \times 3.04 \times 200 \times 0.67^{3/2}$
= 184 m^3/s

Parabolic area of flow over embankment = 2/3 (200 x 1) = 133.4 m^2

V_1 = 1.38 m/s

Flow Over Submerged Weirs

The discharge of a weir is reduced when the downstream water level exceeds the height of the obvert. The submerged discharge can be determined by the following relationship for various types of weir.

$$\frac{Q_s}{Q} = (1 - (H_2 / H_1)^n)^{0.385} \qquad (8.78)$$

where:

- Q = free discharge (unsubmerged), m^3/s
- Q_s = submerged discharge, m^3/s
- H_1 = upstream head, m
- H_2 = downstream head, m
- n = 1.5 for rectangular notch; 2.5 for 90 ^{o} v-notch

Orifice Flow

Orifice flow occurs when a culvert, bridge, sluice gate or pipe opening is fully submerged on the upstream side. The flow occurs due to a pressure head upstream. The general governing equation to estimate the flow through an orifice is given by:

$$Q = C_d * A * (2 * g * H)^{0.5} \qquad (8.79)$$

where:

- Q = discharge, m^3/s
- C_d = coefficient of discharge
- A = area of orifice opening, m^2
- g = acceleration due to gravity, m/s^2
- H = difference in head between the upstream and downstream of the orifice

The value of the discharge coefficient, C_d, can vary significantly depending on the head upstream of the orifice (bridge, culvert, etc.) and whether or not the orifice is submerged (i.e. tailwater elevation is higher than the orifice obvert). Typically, C_d varies between 0.5 to 0.9. To select the appropriate value for application to bridges and culverts refer to the corresponding sections in Chapter 5. A discussion on the application of this equation is also covered in this chapter in the section on culvert hydraulics. (Reference: U.S. Army Corps of Engineers, HEC-RAS User' Manual, 1995).

Hydraulic Models

Several computer programs have been developed to determine water surface profiles for both steady and unsteady gradually varied flow conditions. The purpose is to analyze given hydraulic conditions. Computer programs based on rapidly varied flow conditions or the momentum principle are few and are beyond the scope of this manual.

A summary of the most commonly used computer programs used for hydraulic analysis is provided in Appendix 8A. Users are encouraged to review the documentation for these programs to ensure that the programs are used in the appropriate circumstances. It is the users responsibility to correctly apply any computer program and to verify the validity of the results.

It is usually good practice to:

X read the appropriate manuals for assumptions and limitations of use;
X identify the theory of the model and its applicability to the problem in hand; and
X verify that the accuracy of the results for each application.

Hydraulic Routing

The main purpose of hydraulic routing is calculation of flow rates and water levels at different location of a watershed. Routing is the mathematical expression of the flow of water through the different components of a watershed from the uppermost point in the watershed. Routing may, therefore, be assessed as sheet flow over land, or as flow through a series of channels and reservoirs.

Hydraulic routing procedures are not normally conducted in the design of highway stormsewer drainage systems. It is done normally when upstream water levels and velocity influence downstream conditions. The selection of the most appropriate routing method and computer model is left to the judgement of the analyst/designer.

Flood levels can be calculated using hand calculations or computer programs. Hand calculations generally assume steady, uniform low conditions to determine water levels. Many computer programs that are used to calculate water levels assume steady, gradually varied flow conditions. HEC2 and WSPRO are two computer models for hydraulic routing which are typically used for open channel gradually varied flow conditions. Another computer model is EXTRAN which can be used to solves for unsteady flow conditions in sewers.

Standard Step Method

The standard step method is widely used to determine the water surface profile (depth, velocity etc.) for natural channels and artificial channels. This method applies the principles of continuity and conservation of energy to steady or gradually varied flow. An initial depth and flow rate must be known for a cross section from which the calculation process starts. The method is applicable for subcritical and supercritical flow conditions. For subcritical flow, analysis proceeds in an upstream direction and, for supercritical flow, analysis proceeds downstream. The method is not applicable for rapidly varied flow conditions because losses due to acceleration are not considered. Friction losses are calculated using the Manning equation. Transition losses, due to contraction and expansion of flow are also accounted for.

The standard step method may be used for backwater analysis for natural watercourses. Many computer models, such as HEC2 and WSPRO, incorporate this method.

Example 8.27: Application of the Standard Step Method

Required

Calculate the water surface profiles along a rectangular channel of constant dimensions at two locations, at 300 m and 400 m upstream from a location of known water surface elevation using the standard step method.

Given

Q	= flow rate, m^3/s	= 30 m^3/s
b_w	= channel bottom width, m	= 10 m
n	= Manning roughness coefficient	= 0.025
∀	= energy coefficient	= 1.1

At the Downstream starting point,	
Water surface elevation (WSE), m	= 171.0 m
Y = flow depth, m	= 1.8 m
300 m Upstream	
Channel Invert Elevation	= 169.7 m
400 m Upstream	
Channel Invert Elevation	= 169.9 m

Solution

Step 1 Calculate the following parameters at the downstream starting point, Station 0.0:

Flow area, A (column 4), m^2
Flow velocity, V (column 5), m/s
Velocity head times energy coefficient ∀ (column 6), m
Total energy head, E (column 7), m
Hydraulic radius, R (column 8), m
Friction slope, S_f (column 9), m/m
Total energy head, E (column 14), m

Note: The flow in this example is subcritical [$y > y_c$ (the critical depth)] and therefore, the starting point is downstream. For a supercritical flow condition, the starting point would have to be at the upstream end.

Step 2 Make the first trial estimate of the water surface elevation, WSE at the upstream Station 300. Value is shown in the computation table, column (2).

Step 3 Determine the parameters for columns 3 through 14.

If the total energy head, E in column 7 (determined using the first trial estimate of water surface elevation), is very close to the total energy head, E in column 14 (determined using the average friction slope) then the water surface elevation at Station 300 has been determined.

If the values are not close (within say 0.01 m) then the second trial estimate of the water surface elevation, WSE (column 2) is made and the computations are carried out again. If the total energy head values, E (column 7 and 14), are close, then calculations can proceed to the upstream Station 400.

Step 4 Steps 2 and 3 are repeated for the section at Station 400.

All calculations are shown in the following table.

Standard Step Calculations

STATION (1)	WSE m (2)	y m (3)	A m^2 (4)	V m/s (5)	$\alpha V^2/2g$ m (6)	E m (7)	R m (8)	S_r m/m (9)	S_r m/m (10)	Δx m (11)	h_f m (12)	h_c m (13)	E m (14)
0.0	**171.0**	**1.8**											
	...	**...**	**18.0**	**1.67**	**0.16**	**171.16**	**1.32**	**0.0012**	**....**	**...**	**...**	**...**	**171.16**
300													
Trial #1	**171.5**	**171.5 -169.7 =1.8**	**18.0**	**1.67**	**0.16**	**171.66**	**1.32**	**0.0012**	**0.0012**	**300**	**0.36**	**0.0**	**171.52**
Trial #2	**171.4**	**1.7**	**17.0**	**1.77**	**0.18**	**171.57**	**1.27**	**0.0014**	**0.0013**	**300**	**0.39**	**0.0**	**171.55**
400													
Trial #1	**171.6**	**171.6- 169.9=1.70**	**17.0**	**1.77**	**0.175**	**171.77**	**1.27**	**0.0014**	**0.0014**	**100**	**0.14**	**0.0**	**171.69**
Trial #2	**171.5**	**1.6**	**16.**	**1.88**	**0.195**	**171.70**	**1.21**	**0.0017**	**0.0016**	**100**	**0.16**	**0.0**	**171.71**

Column Description

(1) Station number.

(2) Water surface elevation (WSE). The first WSE is taken at the downstream section.

(3) Flow depth equal to the estimated water surface elevation, WSE (2) minus the channel invert elevation

(4) Flow area based on column (3), m^2.

(5) Mean flow velocity based on Q and column (4), m/s.

(6) Velocity head based on column (5), m.

(7) Total energy head = columns (2) + (6), m.

(8) Hydraulic radius corresponding to column (3), m.

(9) Friction slope, from Manning formula, = $n^2V^2/R^{4/3}$, m/m.

(10) Average slope over the reach equal to average S_f friction between this and previous station, m/m.

(11) Reach length equal to difference of station distances in column (1), m.

(12) Friction loss = column (10) x column (11), m

(13) Eddy loss, m.

(14) Total Energy head equal to total head loss + downstream total energy head columns = (12) + (13) + previous column (14). If column (14) does not closely match column (7), then column (2) is varied until the two columns agree. When column (14) approximately equals column (7), then computations continue with the next upstream station.

Calibration and Verification of Models

Calibration and verification of hydraulic models requires large amount of data including surveyed cross sections, recorded water levels and flow rates.

The calibration and verification procedure involves varying input parameters until a good agreement exists between measured and simulated values. The following parameters are typically varied during calibration:

X channel and flood plain roughness; and
X expansion and contraction coefficients.

It is good practice to carry out a sensitivity analysis of the input parameters, to assess which parameters have the most effect on the results of the model. Input parameters to which a model is sensitive should be selected with great care to achieve sufficient accuracy.

Calibration is the process of finding the most appropriate model parameters so that differences between simulated and observed flows are acceptable. Verification is the process whereby the calibrated model is tested by the use of independent data to check the stability of the model.

The following steps can be used for the calibration and verification process:

- formulate a model based on the most reasonable assumptions;
- adjust the parameters that control the runoff volume to produce a match between simulated and actual values;
- it may be necessary to iteratively do the above steps to achieve a good fit between the
- actual and simulated values;
- fix the parameters and run the model for several data sets and examine the goodness-of-fit

- for observed and simulated hydrographs.

Culvert Hydraulics

Typically, in design applications, culvert hydraulics is used to determine culvert capacity. Laboratory tests and field observations have shown that there are two major types of culvert flow.

- **Flow with inlet control** means that the discharge capacity of a culvert is controlled at the culvert entrance by the depth of headwater and the entrance geometry, including the barrel shape, cross sectional area and the type of inlet edge. The roughness and length of the culvert barrel, and the outlet conditions are not factors in determining the culvert capacity. The longitudinal slope reduces headwater only to a small degree, and can normally be neglected for conventional culverts flowing in inlet control.
- **Flow with outlet control** means that the discharge capacity of a culvert is controlled by the depth of tailwater including the velocity head within the barrel, entrance losses and friction losses. The roughness, length of the culvert barrel, and slope are factors in determining the culvert capacity; the inlet geometry is of lesser importance.

In most cases the operating flow condition of the culvert is not known. The culvert may flow in either inlet or outlet control depending on the flow rates. For this reason, the headwater depth is computed, for the same flow rate, in both the inlet and outlet controls conditions. The higher value indicates the type of control, and should be used as the governing depth in design (refer to Chapter 5 for details). This method is relatively accurate except for the few cases where the headwater is approximately the same for both types of control. Computational procedures are simplified with the use of design nomographs (refer to the Design Charts) and computer programs (see Table 8.5).

In addition to inlet and outlet control, other culvert hydraulic factors discussed in this section are:

- inlet efficiency;
- non-standard roughness coefficients; and
- performance curves for conventional culverts.

Note: For the purpose of this manual, a conventional culvert is a closed invert culvert having a uniform barrel section throughout, no side-tapering and without a slope-tapered inlet. Nearly all existing and most new culverts fall in the conventional culvert category, including those with minor improvements such as bevelled edges, headwalls, wingwalls, or a standard prefabricated end section.

Culverts Flowing in Inlet Control

Sketches of inlet control flow for both unsubmerged and submerged projecting entrances are shown in Figure 8.30 a and b. Figure 8.30 c shows a mitered entrance flowing submerged with inlet control. An increase in barrel slope reduces headwater only to a small degree, and can normally be neglected for conventional culverts flowing in inlet control.

When the headwater (HW) exceeds 1.5D, true orifice flow exists and can be represented by:

$$Q = C_d \sqrt{2g(HW - D/2)} \tag{8.80}$$

where:

C_d = coefficient of discharge (see Table 8.6)
A = cross section area of discharge of the culvert, m^2
g = the acceleration due to gravity, m/s^2
HW = headwater depth, m (refer to Figure 8.30)
D = diameter of the culvert, m

Table 8.6: Inlet Loss Coefficients (C_d)

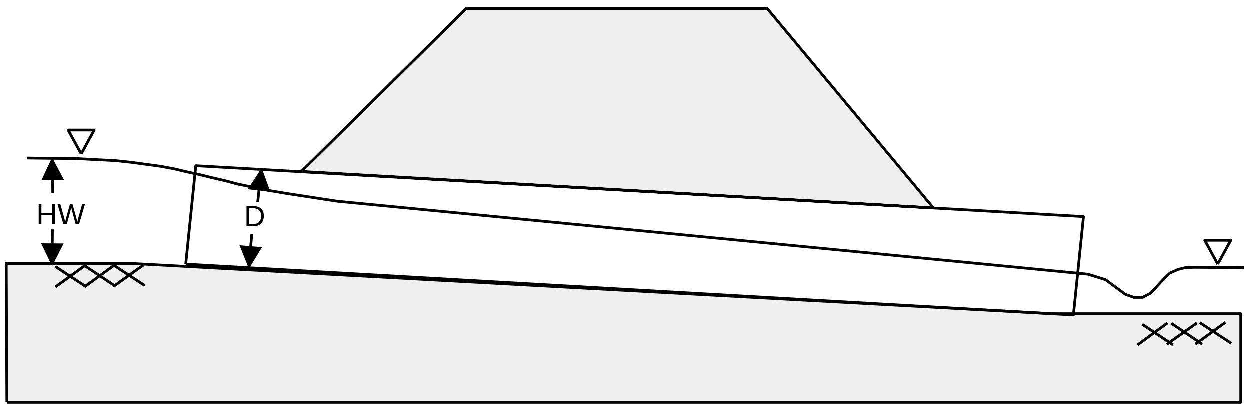
Inlet Type	Discharge Coefficient
Thin Walled Projecting (CSP)	0.50
Flush Headwall	0.60
Cylinder Inlet (1.25 D)	0.67
Socket Inlet (RCP)	0.70
Bellmouth Inlet	0.97

Design Charts 5.39 to 5.45 can be applied in place of Equation 8.80.

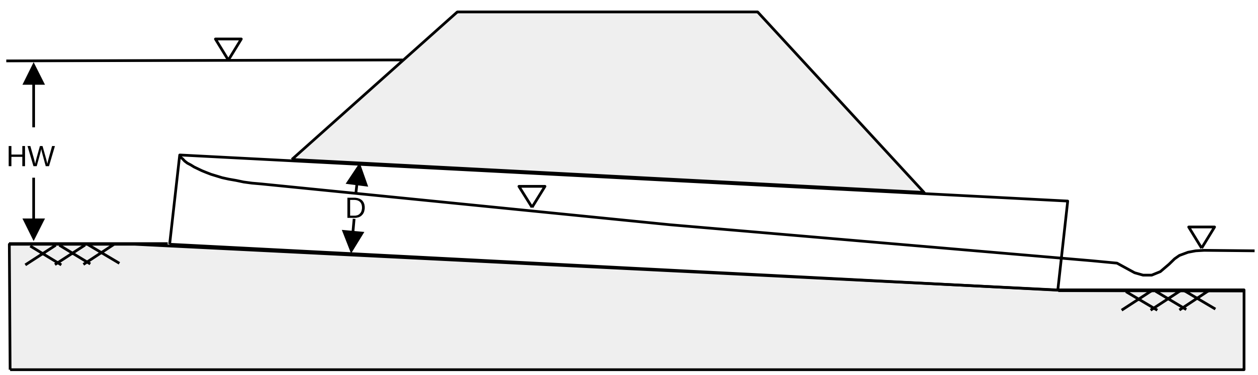
When the headwater HW is less than 1.5D, the culvert acts as a weir with a circular cross section; however, the weir equation cannot be solved analytically for such an application and it is not used in practice. Design Charts 2.31 to 2.33 and 5.39 to 5.45 can be used in such cases.

In all culvert design applications, it is important to recognize that headwater, or depth of ponding at the entrance to a culvert, is an important factor in culvert capacity. The headwater depth (HW) is the vertical distance from the culvert invert at the entrance to the energy gradeline of the headwater pool (depth + velocity head - refer to Figure 8.30). Because of the low velocities in most entrance pools, the water surface and the energy line at the entrance are usually assumed to be coincident, thus the headwater depths given by the inlet control charts (Design Charts 2.31 to 2.33 and 5.59 to 5.46) will be higher than will actually occur, by the amount of the velocity head $V^2/2g$ (refer to Figure 8.32). The difference may be ignored unless the approach velocity V_1 is exceptionally high.

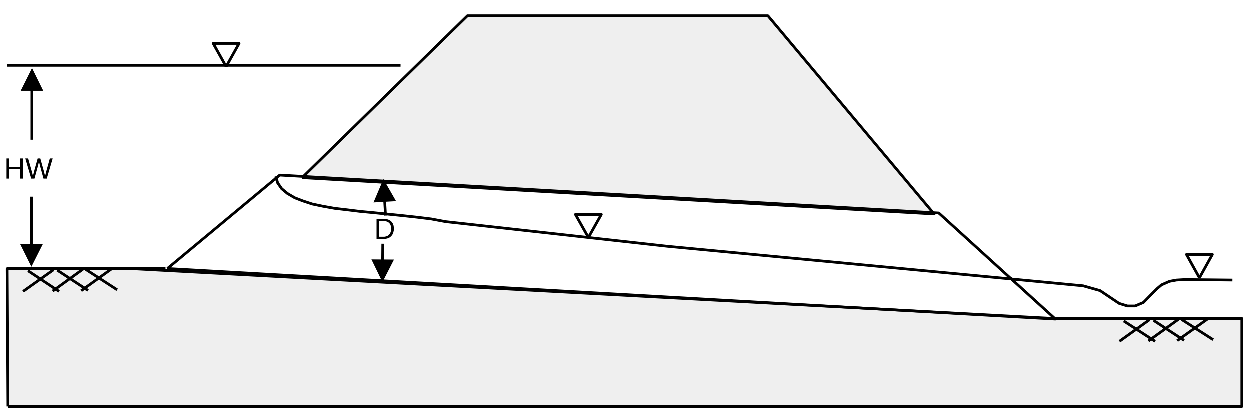
Figure 8.30: Flow Profiles for Culvert in Inlet Control



a. Projected End - Unsubmerged Inlet

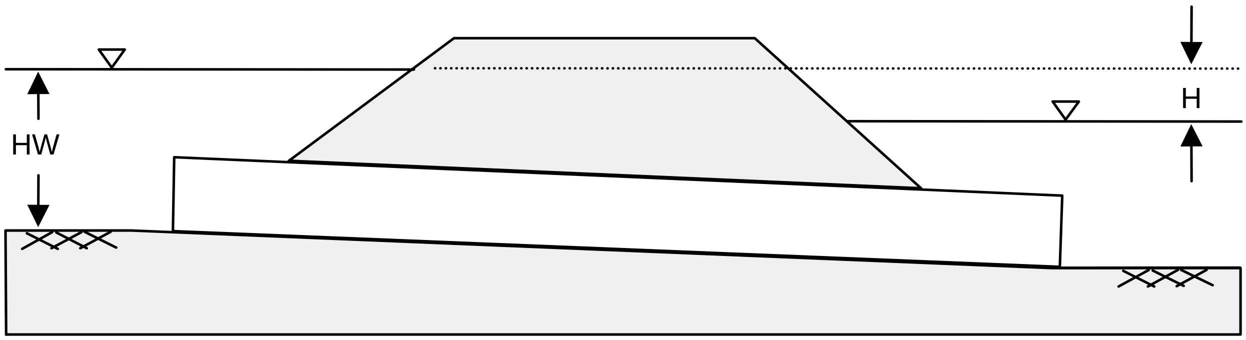


b. Projected End - Submerged Inlet

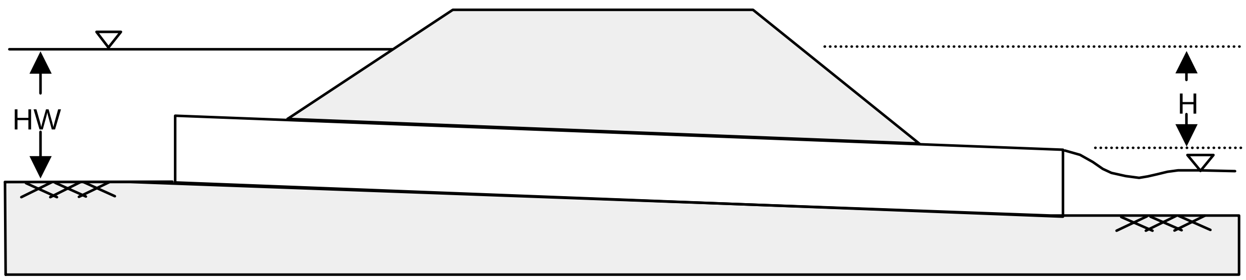


c. Mitered End - Submerged Inlet

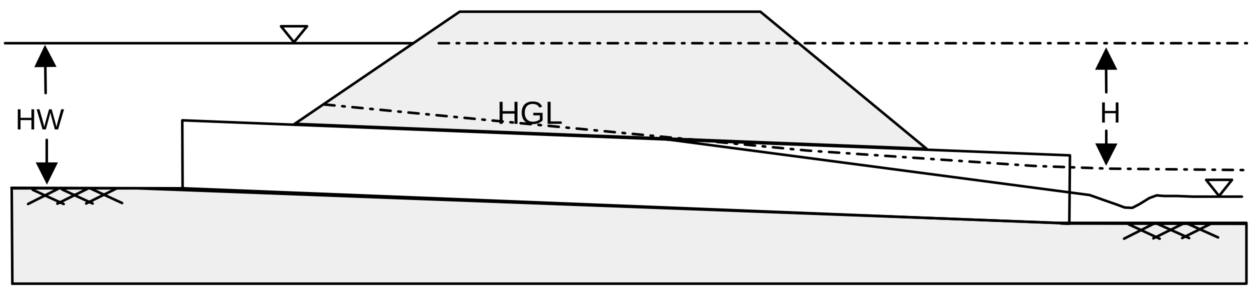
Figure 8.31: Flow Profiles for Culvert in Outlet Control



a. Culvert Flowing Full, Submerged Outlet



b. Culvert Flowing Full, Unsubmerged Outlet



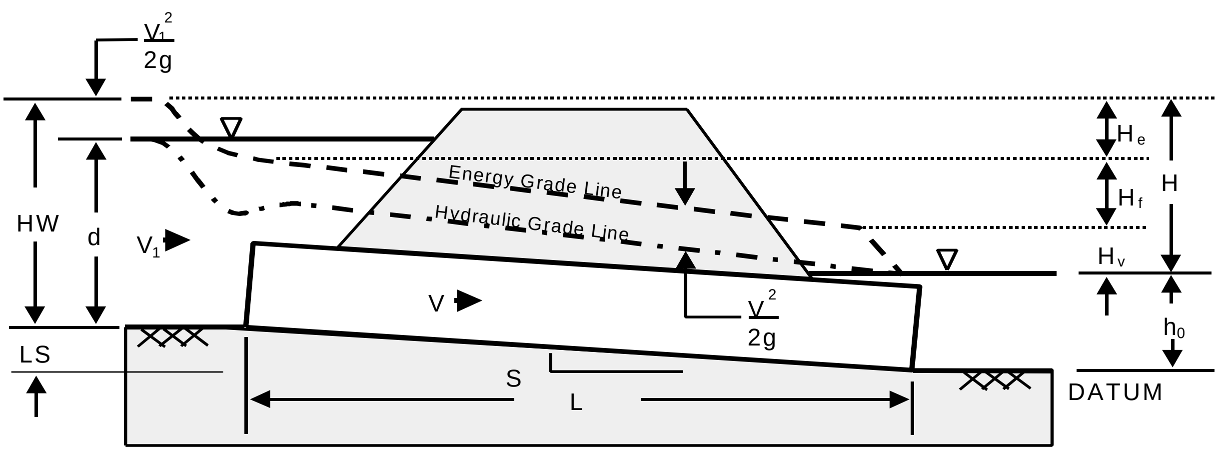
c. Culvert Flowing Full For Part Of Its Length

d. Culvert Not Flowing Full

Culverts Flowing in Outlet Control

Culverts in outlet control can flow with the culvert barrel full or partly full (see Figure 8.31). If the entire cross section of the barrel is filled with water for the total length of the barrel, the culvert is said to be flowing full, Figure 8.31 a and b. Two other common types of outlet control flow are shown in Figures 8.31c and d.

Figure 8.32: Hydraulics of Culvert Flowing Full in Outlet Control



The expression for determining the head H is derived by equating the total energy upstream the culvert entrance to the energy just inside the culvert outlet, considering all the major losses in energy. By referring to Figure 8.32 and using the culvert invert at the outlet as a datum, the result is:

$$d + \frac{V_1^2}{2g} + LS = h_0 + H_v + H_e + H_f \qquad (8.81)$$

where:

- d = depth of flow at inlet, m
- $V_1^2/2g$ = velocity head in entrance pool, m
- LS = length of culvert multiplied by the culvert barrel slope, m
- h_0 = depth of flow at outlet, m
- H_v = the velocity head within the barrel, m
- H_e = the entrance loss, m
- H_f = the friction loss, m

Rearranging Equation 8.81:

$$d + \frac{V_1^2}{2g} + LS - h_0 = H_v + H_e + H_f \, .$$

Head (H)

Head (H) is expressed as an equivalent depth of water (m), and comprises the velocity head within the barrel H_v, the entrance loss H_e, and the friction loss H_f (refer to Figure 8.32),

$$H = H_v + H_e + H_f \,. \qquad (8.82)$$

The velocity head is the difference in elevations between the energy grade line and the hydraulic grade line, which are parallel over the length of the barrel except in the immediate vicinity of the inlet, where the flow contracts and then expands (refer to Figure 8.32). The velocity head H_v is:

$$H_v = \frac{V^2}{2g} \qquad (8.83)$$

where:

- V = the mean velocity in the culvert barrel, m/s (the mean velocity is the discharge Q, divided by the barrel cross sectional area A)
- g = acceleration due to gravity, m/s^2

H_e accounts for entrance losses and depends upon the geometry of the inlet edge. Lost energy is expressed as a coefficient k_e times the barrel velocity head, or:

$$H_e = k_e \frac{V^2}{2g} \qquad (8.56)$$

where:

- k_e = entrance loss coefficients (Design Chart 2.08).

The friction loss H_f is the energy required to overcome the roughness of the culvert barrel. The friction loss is given by Equation 8.57, which is derived from the Manning equation.

$$H_f = \frac{19.6\, n^2 L}{R^{4/3}} \frac{V^2}{2g} \qquad (8.57)$$

where:

- n = Manning's roughness coefficient (Design Chart 2.01);
- R = hydraulic radius, m
- L = length of culvert barrel, m

Substituting H_v, H_e and H_f in Equation 8.83 and simplifying, the head (H) for full flow is:

$$H = \left[1 + k_e + \frac{19.6\, n^2 L}{R^{4/3}} \right] \frac{V^2}{2g} \qquad (8.84)$$

Equation 8.84 can be solved for H by use of the full-flow nomographs, Design Charts 2.34 to 2.36 and 5.47 to 5.50. Each nomograph is drawn for a particular barrel shape and material, and for the roughness coefficient noted on the respective charts. These nomographs can be used for other values of n by modifying the culvert length (refer to the section on non standard roughness coefficients).

By substituting H for $H_v + H_e + H_f$, Equation 8.81 can be simplified:

$$d + \frac{V_1^2}{2g} + LS - h_0 = H . \tag{8.85}$$

Headwater Depth (HW)

It can be seen from Figure 8.32 that the velocity head (i.e. $V_1^2/2g$) at the culvert entrance, is the difference between the elevations of the hydraulic grade line at the inlet and the energy line at the inlet. Because of the low velocities in most entrance pools, the water surface and the energy line at the entrance are usually assumed to be coincident. In culvert design, the difference may be ignored unless the approach velocity V_1 is exceptionally high; $V_1^2/2g$ can be assumed to be equal to zero, and, as a result, d will equal HW (refer to Figure 8.32). Substituting into Equation 8.85:

$$HW + LS - h_0 = H . \tag{8.86}$$

Determination of h_o

A detailed explanation on the determination of h_0 is provided in Smith C.D, (1985) or U.S. FHWA (1985); for convenience, a summary is provided below.

Tailwater at or Above Top of Opening

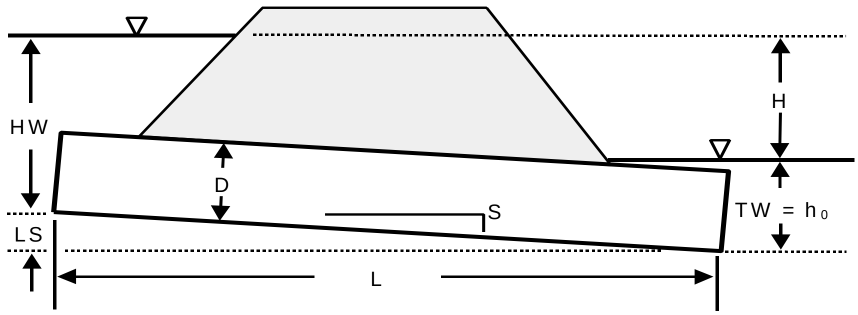
When the water surface in the outlet channel is at or above the top of the culvert outlet (refer to Figure 8.33) h_o is equal to the tailwater depth. Tailwater depth, TW, is the depth from the culvert invert at the outlet, to the water surface in the outlet channel.

Tailwater Below Top of Opening

If the tailwater elevation is below the top of the culvert opening at the outlet, as in Figures 8.31 b, c and d, h_o is more difficult to determine. In these cases, h_o is the greater of two values:

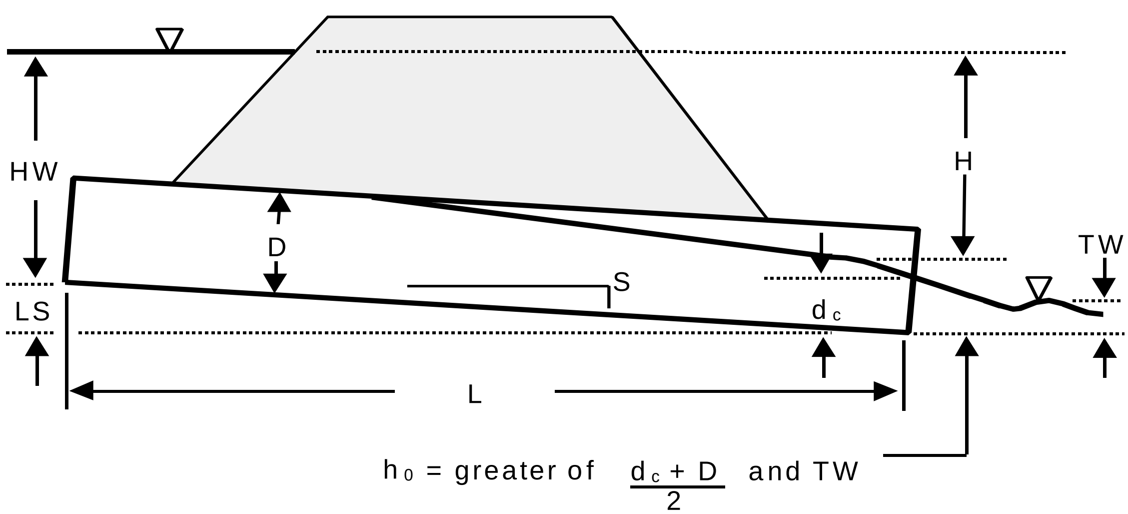
1) TW depth as defined above; or
2) $(d_c + D)/2$.

Figure 8.33: Determination of h_o for High Tailwater



The latter dimension is the height from the invert to the equivalent hydraulic grade line (located at half the height between critical depth and the culvert crown). The critical depth d_c may be obtained from Design Charts 2.37 to 2.38, and 5.50 to 5.54, where D is the culvert depth. The value of d_c can never exceed D, making the upper limit of this fraction equal to D. Where TW is the greater of values (1) and (2) above, the critical depth is submerged sufficiently to make TW effective in increasing the headwater. Figure 8.34 shows the terms of Equation 8.86 for the low tailwater condition. A change of discharge can modify the water surface profile to that of Figure 8.31 b or d. In the latter case accuracy of the results diminishes as HW approaches 0.75 D; for smaller values of HW detailed backwater computations should be used, as necessary.

Figure 8.34: Determination of ho for Tailwater Below Top of Opening



Summary

Rearranging Equation 8.85, the headwater depth can isolated so that HW can be determine from one equation for all outlet control conditions:

$$HW = H + h_0 - LS \tag{8.86}$$

where:

H = downstream head, m
(Design Charts 2.34 to 2.36 and 5.47 to 5.50 or from Equation 8.84;

h_o = greater of TW and d_c + D/2 in which dc_ D, m
(see following discussion and Figure 8.34);

D = culvert height, m

d_c = critical depth from Design Charts 2.37 to 2.38 and 5.51 to 5.54, m

S = slope of culvert barrel, m/m

L = length of culvert barrel, m

Inlet Efficiency

The capacity of a culvert operating in inlet control can be significantly increased by providing an efficient inlet, which reduces the flow contraction at the entrance and increases the flow depth in the barrel. The relative efficiencies of various inlet shapes can be judged from the entrance loss coefficients, k_e, in Design Chart 2.08, although the figures are not directly applicable to inlet control. Inlet improvements and end treatments are discussed in Chapter 5.

In outlet control, entrance losses form only a minor part of the total head losses, and major inlet improvements are not usually justified.

Thin Edged Inlets

Projecting thin edged inlets on steel culverts are relatively inefficient (k_e = 0.9), but are very widely used, for culvert spans of less than 3.0 m, because of their simplicity and lower cost. Standard end sections for small culverts improve their efficiency (k_e = 0.5), and properly bevelled concrete collars provide the greatest improvement (k_e = 0.25) but are costly. Culvert ends mitered to conform with the fill slope offer a slight hydraulic improvement, (k_e = 0.7).

Bevelled Inlets

Bevelled inlets increase the efficiency of concrete culverts (k_e = 0.2) and corrugated steel culverts (k_e = 0.25).

Socket Ends

Socket ends of concrete and plastic pipes are hydraulically efficient (k_e = 0.2); therefore, the inlets of these pipes may not need to be mitered or cut on skew.

Headwalls and Wingwalls

Headwalls and wingwalls improve inlet efficiency for some types of culvert, as indicated by the entrance loss coefficients in Design Chart 2.08.

Non-standard Roughness Coefficients

Roughness Coefficient Other Than Nomograph Value

If the culvert has a roughness coefficient different from that of the outlet control nomograph, as in the case of a timber box culvert, the culvert length should be adjusted to compensate for different n-values before entering the nomograph, as follows:

$$L_1 = L \left[\frac{n_1}{n} \right]^2 \tag{8.87}$$

where:

- L = actual length of culvert, m
- L_1 = adjusted length of culvert, m
- n = roughness coefficient on which the outlet control chart is based
- n_1 = actual roughness coefficient.

Composite Roughness Coefficient

Where the culvert perimeter has differing roughness coefficients, such as a corrugated steel arch with a concrete floor, the overall coefficient, n_c, is found as follows:

$$n_c = \frac{[P_1 n_1^2 + P_2 n_2^2 + ...]^{0.5}}{P^{0.5}} \tag{8.88}$$

where:

- P_1, P_2 = wetted perimeter having roughness coefficients of n_1, n_2... , m
- n_1, n_2 = roughness coefficients
- P = total wetted perimeter of culvert, m

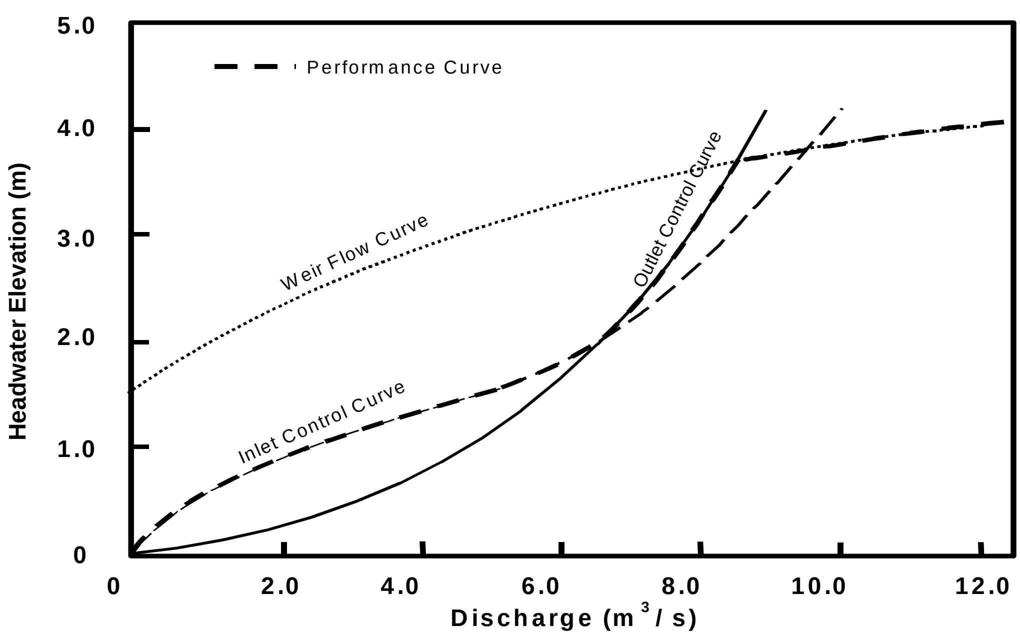
Performance Curves for Conventional Culverts

To understand how a culvert will function over a range of discharges, a performance curve, which is a plot of discharge versus headwater elevation, may be drawn. Major components of performance curves are:

- inlet performance (i.e. inlet control);
- outlet performance (i.e. outlet control); and
- roadway spill (i.e. weir flow);

A typical overall performance curve is shown in Figure 8.35. As shown, the overall performance curve is a combination of the inlet, outlet and roadway overtopping (weir flow) that best corresponds with the hydraulics of the culvert with relief.

Figure 8.35: Typical Overall Performance Curve



Performance curves for each alternative culvert size, type and entrance geometry can be developed to assist in the selection of the most appropriate design.

A minimum performance is usually assumed in culvert designs as a safety against uncertainties in flood estimation etc. The minimum performance is taken as the discharge corresponding to the highest water level as determined from an inlet and outlet analysis. In this way, the culvert will not operate at a lower level of performance than was calculated.

Soil Loss Calculations

In general, to assess the type and specification of sediment and erosion control measures it is necessary sometimes to determined the amount of soil being transported from a development site. However, calculation of soil loss is included in this manual to provide specific guidance on the calculation of sediment storage requirements for sedimentation basins on construction sites. The information included here is in support of the sedimentation basin design example in chapter 6.

There are two established methods for the calculation of soil loss, the universal soil loss equation (USLE) and the modified universal soil loss equation (MUSLE). The use of other methods may be possible, however, their suitability for application to highway construction sites, under Ontario conditions should be established before proceeding with a design.

Universal Soil Loss Equation (USLE)

According to the publication *Erosion Control During Highway Construction* (U.S. Transportation Research Board, 1980) the universal soil loss equation (USLE) developed by the Agricultural Research Service (1980) is a suitable method for predicting soil loss caused by sheet and rill erosion during highway construction. The equation was developed by Wischmeier and others (1978) for estimating soil losses from farm lands east of the Rocky Mountains, and has since been modified and extended to make it applicable to highway construction sites. A 1980 research report indicated that 21 states in the U.S. were using the equation (Israelson, 1980). The same publication reported the results of tests indicating that the USLE is valid for slopes as steep as 84% (1.2:1), as well as for flatter terrain, thus confirming its applicability to highway cut and fill slopes.

To confirm the validity of the USLE for highway conditions in Ontario, research was carried out by Dr. G. Wall of the University of Guelph to determine the appropriate soil erodibility and rainfall factors for use in the equation. The results have been incorporated in the appropriate design charts included in Part 4 of this manual..

The USLE has some limitations which the user should be aware of.

1. The equation predicts long-term average annual soil loss rates, which may differ greatly from observed values for individual storms or periods. The user should, therefore, be aware that the calculations at best give only an indication of the erosion potential in a given situation, and not a precise estimate of the actual soil loss.

2. The equation estimates soil losses caused by rainfall and not by thaw or snowmelt. The effects of thaw and snowmelt may be allowed for in the procedures given herein by modifying the rainfall and erodability factors.

3. The equation accounts for only sheet and rill erosion. Gully and channel erosion caused by concentrated flows should be prevented by adequate control of overland flow and proper design of permanent drainage facilities.

4. The equation does not account for transportation of sediment beyond the toe of slope.

The metric form of the USLE is:

$$E = 2.24\ R\ K\ LS\ VM \tag{8.89}$$

where:

E = mean annual soil loss rate, t/ha
(E may also be calculated for a period other than a year)
R = mean annual rainfall factor (see discussion below on R factor regarding units)
K = soil irritability factor (see discussion below on K factor regarding units)
LS = topographic factor, dimensionless
VM = erosion control factor (or vegetative-mechanical factor), dimensionless

These factors are discussed in the following paragraphs. The coefficient 2.24 converts tons/acre to tonnes/hectare.

The quantity (mass) of material E_A (in tonnes) moved from an area A (hectares) is given by:

$$E_A = E*A \tag{8.90}$$

The volume of material E_V (in cubic metres) moved is given by:

$$E_V = (1/\rho) / E_A \tag{8.91}$$

where :

ρ = density of eroded soil, kg/m^3.

If the density is not known, an average value of 1.0 t/m^3 may be used for predominantly mineral sediment.

Of the four components of the USLE, only LS and VM can be modified significantly, and even then only on some projects. Occasionally R may be reduced by varying the time or duration of construction, and K by realigning the highway to avoid highly erodible soil or by using less erodible fill material. Since the factors are multiplicative, even small changes can significantly affect the soil loss.

Rainfall Factor R

The rainfall factor R is a measure of the average annual erosive force exerted by storm rainfalls occurring over a long period of records. The factor may be calculated from recorded rainfall intensity and duration data by the method given in Appendix E of the document *Erosion Control During Highway Construction* (Israelson, et al, 1980). The R factor has the English unit of (100ft tons/acre * in/hr). See discussion on the K factor for the rationale for using English units.

Broadly speaking, areas having high R values are subject to higher erosion rates than those with low values. The curves in Design Chart 6.03, developed by Dickeson and Wall, give average annual R values for Ontario (with approximately 2-year return period), using a modification of the method of Wischmeier and Smith (1978). The values range from less than 50 in northern Ontario to over 125 in extreme southwestern Ontario. (It should be noted that R has been kept in imperial units to avoid the need for using new values for K). The R values include a nominal allowance for the effects of snowmelt and thaw, although in highly thaw-susceptible soils, an additional allowance may be made by adjusting K as described in the next subsection.

The distribution of R values over periods less than one year may be found from the table in Design Chart 6.03. This table is used for estimating erosion during specified construction periods.

Soil Erodibility Factor K

The soil erodibility factor is a measure of the tendency of a soil to erode as a result of the energy of rain drops. As derived, it is independent of slope and surface treatment, which are accounted for by the LS and VM f actors respectively. Highly erodible silty soils may have a K factor exceeding 0.6 while relatively inerodible sandy soils may have a factor less than 0.2.

As noted above, the use of metric R values would necessitate converting all existing values of K. To avoid possible confusion arising from a new set of K values, R and K have been kept in imperial units and the calculated soil loss converted to metric units by means of the factor 2.24. For a detailed discussion of the complexities of fully metricating the USLE, see the Appendix in the publication by Wischmeier and Smith (1978).

The erodibility of a soil depends on the particle size and distribution, soil structure, void space, pore size, and amount of organic material. Usually the erodibility is greater for silty soils and is reduced by the presence of organic material such as roots and other debris. Subsoils generally lack significant organic content.

The erodibility of some soils may increase considerably during periods of thawing when the subsoil is still frozen. A study by Dickinson and others (1982) indicates that this effect is most pronounced for predominantly clay soils and least for loam soils. The reported increases for predominantly sand soils are highly variable, but are generally greater than for loams. Design Chart 6.01 (Thaw Factor), which is based on the study by Dickinson et al (1982), groups soils into predominantly sand, loam and clay, and lists factors which may be applied to K in the early spring months when surface thawing occurs. In the case of sand soils, two extremely high values are excluded from the estimation of the median, since the laboratory measurements indicate a much lower value. The effect of thawing is allowed for to some extent in the R factor, therefore, the thaw factor need be applied to the K value only in critical cases.

K factors for soils on MTO projects may be provided by MTO. If the information is not available, one of the following approaches may be used.

Wischmeier Nomograph

Design Chart 6.02 is a nomograph developed by Wischmeier and others (1971) for determining K if a detailed soil analysis is available The procedure is illustrated on the nomograph.

According to Roth and others (1974), the Wischmeier nomograph lacks the desired sensitivity to differences in erodibility in exposed B and C horizons in clay subsoils. For such soils the content of free iron and aluminum oxides ranks next to particle size distribution as an indicator of erodibility.

K Values Related to Soil Textures

Design Chart 6.01 enables the user to determine K from the soil texture (e.g. silt loam) if a particle size analysis is not available.

Soil textures may be determined in several ways.

(a) Textural classification charts may be used to determine soil textures if the approximate percentages of sand, silt and clay are known.

(b) The MTO document *Guide for Soils Field Inspectors* (Fig. 3-3), describes simple field tests for determining soil textures.

(c) County soil reports (20) may be useful in cases where no other information is available, since some of them contain information on subsoils as well as surface soils.

K Values for Surface Soils

A list of K values for selected surface soil types (e.g. Huron silt loam) has been prepared by the University of Guelph. The list has not been included in the chapter because of its bulk and the fact that it refers only to surface soils, and would therefore be rarely applicable to highway construction.

The overall weighted K for a continuous slope consisting of significantly differing soil types is:

$$K = \frac{K_1A_1 + K_2A_2 + ...}{A_1 + A_2 + ...} \tag{8.92}$$

where:

K_1, K_2 = erodibility factors obtained (Design Chart 6.01 or 6.02)

A_1, A_2 = inclined face areas of surfaces 1 and 2 respectively (not the plan areas).

Segments of discontinuous slopes are treated as individual units.

Topographic Factor LS

The topographic factor LS represents the combined effect of slope length L and gradient S on soil erosion, and provides the designer with a means of comparing various combinations of L and S. It should be noted that the factor is not calculated simply by multiplying L and S, but must be obtained from Design Chart 6.4 (Foster and Wischmeier, 1973). The LS factors were originally based on studies of slopes up to 20% or 5:1, but since then, their validity has been confirmed for slopes up to 84% or approximately 1.2:1 (Israelson, 1980).

It is obvious that short flat slopes will erode less than steeper longer slopes. However, the effect of flattening a slope of a given height is not so obvious, since at the same time the slope length will be increased. For example, flattening a 1.5:1 slope 30 m long to 3:1 would reduce LS from approximately 26 to 12, thereby reducing the erosion rate per hectare by 54%, but the erosion quantity (rate x area) is reduced by only 17% because of the increased slope length (and therefore area).

On the other hand, halving the slope length (e.g. by providing a bench), reduces the erosion quantity by approximately 30%.

Erosion Control Factor VM

The erosion control factor VM, sometimes known as the cover or vegetative-mechanical factor, characterizes the effects of vegetative and non-vegetative methods of controlling erosion. It does not include the effects of features such as interceptor ditches and slope benches, which are accounted for in the topographic factor LS.

Construction operations which remove all vegetation and the root zone of the soil leave the surface completely without protection, a highly erodible condition corresponding to VM = 1 or other appropriate values indicated by Design Chart 6.05. Vegetative cover reduces erosion by intercepting raindrops and softening their impact, giving very low VM values such as 0.01 for well established grass. Mulch has a similar effect, and has the added benefits of slowing down the runoff, protecting the ground surface until the vegetation becomes established, and assisting germination and growth. The VM factor for a given construction area changes during the course of the work.

The Modified Universal Soil Loss Equation (MUSLE)

William (1975) modified the USLE by relating soil loss to runoff energy rather than the energy of raindrops. The underlying rationale is that the sediment sheared due to rain drop energy may not necessarily travel far without energy being exerted by overland flows. Linking soil loss to runoff also has the advantage of having the assessment based on an individual storm flow rather than a yearly average, as is the case with the USLE.

The rationale provides a better representation of Ontario climate conditions. Precipitation in winter is in the form of snow, which may not result in runoff, then in spring the snow slowly melts resulting in high runoff. As a result, the linkage of soil loss to runoff seems more appropriate to cover the scenarios of year-round conditions.

William's prediction equation, the Modified Universal Soil Loss Equation, in metric units is:

$$S = 2293 (V * Q_p)^{0.56} * K * LS * VM \qquad (8.93)$$

where:
- S = sediment yield, tonnes
- Q_p = peak flow rate period for a selected recurrence period, m^3/s
- V = total volume of flow for a known recurrence period, ha-m

The variables K, LS and VM are the same as described above, for the USLE.

The Total volume of flow, V, can be calculated based on the following equation:

$$V = 0.4762\, Q_p * t_p \qquad (8.94)$$

$$t_p = 0.7 * t_c \qquad (8.95)$$

where:
- t_p = time to peak, h
- Q_p = peak flow rate period for a selected recurrence period, m^3/s
- t_c = time of concentration, h

Sediment Delivery Calculations

Much of the sediment produced by erosion travels only a short distance before it is deposited. This may occur at a flattening of the gradient, at an increase of the resistance to flow such as that caused by vegetation, at the point of inflow into a larger body of water, or at an obstruction such as a silt barrier. The amount of sediment reaching a sensitive area may therefore be significantly lower than the estimated soil loss.

The ratio of sediment delivered to a given point to the quantity eroded is termed the sediment delivery ratio (SDR). This ratio closely approaches 1.0 if the point of delivery is immediately downstream from the eroding area, and is very small if the flow passes through a substantial buffer of dense vegetation or forest litter.

Water-borne sediment may be transported by either sheet flow or channel flow. Factors strongly influencing transportation by sheet flow include the distance travelled, the nature of the terrain and the particle size. In general, the SDR for sheet flow is much lower than for channel flow over the same distance, particularly if the ground is heavily vegetated. Very little information is available on SDRs for sheet flow, but until the state of knowledge improves, the information in Design Chart 6.06 may be used as a rough guide. This is based on preliminary information supplied by Drs. W.T. Dickinson and G.J. Wall.

Stormwater Quality

General Background

The information presented in this section is provided as a general reference to complement the information in Chapters 3 and 4. This information is not necessary applicable directly to highways in all cases, but it provides some insight into the issue of water quality and presents some of the research in the field of water quality control.

Environmental Concern Associated with Stormwater Quality

Prior to the 1980's, the principal efforts made to protect and enhance the quality of the water resources focused on control and treatment of sanitary sewage and industrial wastewater discharges (sometimes referred to as point sources). These efforts are still on-going. However, over time point sources were found not to be the only source of contamination of natural water resources. Stormwater runoff (referred to as non-point source) was found to be a significant contributor to the degradation of water quality. The following are a few cases to illustrate the attention drawn to stormwater quality concerns.

Chesapeake Bay

Chesapeake Bay is situated on the coast of Maryland, Virginia. It is a rich marine fishery resource. In the 1980's, it became increasingly clear that the money spent to control point sources will be virtually wasted unless agricultural and urban stormwater sources of nutrients are controlled. The States of Virginia, Maryland and Pennsylvania have initiated aggressive stormwater runoff control programs aimed at reducing pollution in Chesapeake Bay (Carter, 1985).

Toronto Area Watershed Management Strategy

A study report of this project has concluded that "for most of the conventional water quality parameters, concentrations (in the Humber river) were highest during the wet weather events and lowest during the dry events. Spring runoff concentrations were usually intermediate." (Toronto Area Watershed Management Strategy Steering Committee, 1984).

Lake Simcoe Environmental Management Strategy (LSEMS)

The water quality in Lake Simcoe has been deteriorating over time. One of the reason for the decline in water quality has been attributed to the phosphorus loading to the lake. Phosphorus enters the lake directly from sewage treatment plants, storm sewers and direct runoff from rural lands and indirectly from a number of rivers and streams that transport phosphorus from agricultural lands and urban areas further upstream of the watershed. The 1995 LSEMS progress report provides an estimate of phosphorus loading to Lake Simcoe from the different sources in the lake watershed. Table 8.7 shows a breakdown of the estimated average annual loading of phosphorus, based on 1990 results. It can be seen that in urban areas the loading from stormwater is about three times the load from sewage treatment plants.

Table 8.7: Phosphorus Loading to Lake Simcoe from Different Sources

Source of Phosphorus	Estimated Phosphorus Loading		Remarks
	Metric Tonnes	% of total load	
Septic systems	2.2	2.1	
Sewage treatment plants	5.6	5.4	
Urban dry weather	6.2	6.0	Leaks into storm sewers
Urban stormwater	18.5	17.8	
Live stock	14.5	14.0	
Erosion (from sediment):			
Cultivated land	24.0	23.1	
Pasture/fallow	6.5	6.3	
Forests	8.0	7.7	
Wetlands	4.5	4.3	
Idle/ scrub lands	3.0	2.8	
Atmospheric deposition	11.0	10.5	
Total load	104	100	

How to Approach Stormwater Quality Management

Management of stormwater quality is more an applied art than an established physical science. It is difficult to approach stormwater quality concepts and hypotheses in the same manner as those of other engineering sciences such as structural theories or principles of highway geometric design. This can be attributed to a number of factors, some of which are:

- Stormwater quality is a product of natural phenomena involving, among others, watershed response to precipitation, storm behaviour, wind effect. None of these phenomena can be satisfactorily defined by a single law of physics or statistical equation. The combined effects of these phenomena is even harder to define mathematically.

- The same may be said of stormwater quality data. The degree of precision and reliability of stormwater quality data, and results of numerical analysis, cannot be equated with the data and results associated with other engineering disciplines such as testing of engineering materials or analysis for a structural design. In most cases, the properties of engineering materials are reproducible, while those for stormwater quality, even under the same site conditions, are hardly reproducible.
- It is difficult to establish many cause-effect relationships of stormwater quality and long term environmental impacts. One reason for the difficulty may be that biological impacts may not become apparent until years later. Another reason may be that a relationship may not be just one-to-one but may also be many-to-many.
- The mitigative measures developed for treatment of stormwater, in many cases, are based on experimental investigations reflecting specific site conditions. Therefore, the solutions developed are not transportable without further investigations. Mitigative measures developed in a southern state in the United States may not be effective in Ontario and vice versa.

Therefore, when dealing with stormwater quality information, it is prudent to:

- Evaluate the information before use.
- Compare the information in hand with the information from other similar situations.
- Understand the inherent uncertainties in the information and make appropriate allowances.
- Use any data within the scope of the intended use. For example, if the data were collected for determining whether a certain contaminant, say, lead, exists, it would be erroneous to use the data to estimate lead loading from stormwater flow.

The needs to consult and work with professionals of other disciplines and their manuals cannot be over emphasized. For example, the participation of biologists is essential when dealing with aquatic and terrestrial habitat issues, and hydrogeologists when dealing with groundwater issues.

Stormwater Contaminant Types, Sources and Magnitudes

When stormwater quality is a concern, or when checking if a certain stormwater flow may cause a potential water quality concern, it is necessary to determine the types, sources and magnitude of the contaminants in the stormwater flow.

Stormwater Contaminant Types

A comprehensive discussion on the types of contaminants and the associated pollution problems is presented in a publication by Novotny, V. (1981). The following is a summary of the discussion concerning the most frequently mentioned contaminants.

Sediment

Sediment, especially the fine fractions, cut down light penetration to deeper waters resulting in a reduction in algal growth. This has harmful effects on plants and animals living on the bottom of streams and lakes and also harmful to fish, especially to eggs and gill movement. The harm done to fish through destruction of food supply, eggs or changes in the habitat probably occurs long before any adult fish can be directly harmed. Sediment also impairs most beneficial uses of water.

The question of how much suspended sediment will become harmful to surface water cannot be precisely indicated.

Dissolved Oxygen

Dissolved oxygen (DO) concentration of surface water is a primary parameter for determining suitability of water for fish and wildlife. A high DO concentration is generally preferred. Dissolved oxygen is consumed when organic materials in the water decompose. This biological oxygen demand (BOD) is used as a measure of availability of biological matter in water.

Nutrients

Although many elements and chemical compounds are essential for plant and algal growth, only nitrogen and phosphorus are considered the limiting nutrients controlling their growth. When the supply of these nutrients is high, excessive algal growth may occur in a receiving water body. When the algae die and decay, they sink to the bottom of the lake and consume dissolved oxygen resulting, ultimately, in anoxic conditions in the bottom of the lake.

Toxic Metals

PLUARG (*Pollution from Land Use Activities Reference Group of the International Joint Commission*) has determined that mercury, lead, arsenic, cadmium, copper, zinc and chromium should be considered potential hazardous, requiring further attention. Table 8.8 briefly indicates the areas of concern.

Biological Availability of Contaminants

A notable portion of contaminants is carried by suspended solid loads. Contaminants in solid form are not readily usable by living organisms, but contaminants in solution form are.

An International Joint Commission study has found that the fractions of contaminant input to the Great Lakes in solid form are: phosphorus (75%), nitrogen (up to 10%) and heavy metals (55% to

75%). It should be mentioned that the solid-dissolved ratio may vary with the acidity of the water. Generally, when the acidity increases (pH decreases), the soluble fraction increases. Moreover, the concentration of the toxic fraction of some of the nutrients compounds, especially ammonia, increases with lower pH.

Table 8.8: Some Toxic Metals That May Be Found in Surface Waters

Metal	Alleged Effects	Aquatic Life Criteria
Arsenic	Chronic, cumulative, carcinogenic, cardiovascular effects.	0.04 mg/L
Cadmium	Arteriosclerosis, cancer, itai-itai disease.	0.01-1.2 µg/L
Chromium	Cancer, ulceration.	0.29 µg/L
Lead	Cumulative, plumbism.	0.1-3 µg/L
Mercury	Methylated cause, Minamata disease.	0.0005 µg/L
Zinc	Taste problem.	47 µg/L

Source: Novotny V. and G. Chesters (1981).

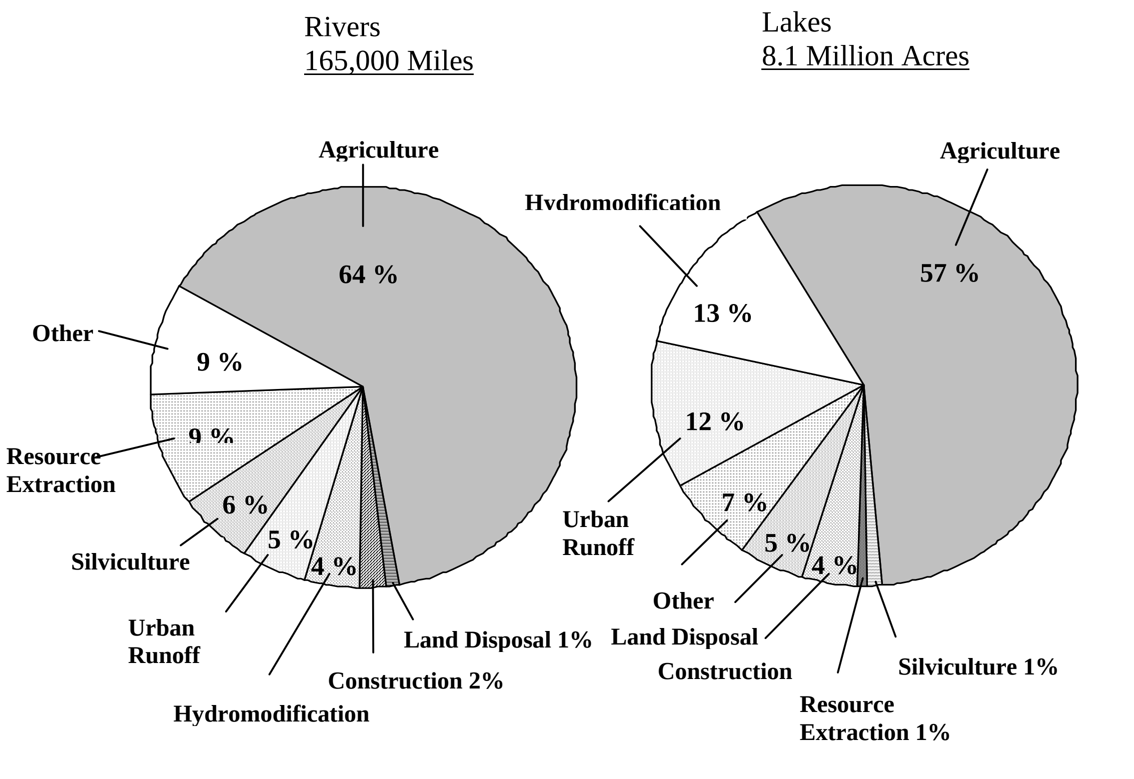
Stormwater Contaminant Sources and Magnitudes

The source and magnitude of a contaminant in a stormwater flow are often stated together to provide a more meaningful picture of the contaminant. There are published study reports on contaminant sources and magnitudes. The scope of these reports ranges from national to local studies. Some of the information provided in these reports is presented here to give a "quantitative" example of contaminants in stormwater. The data are not meant to be used as design data, although they can be used for comparison with project specific data, if necessary.

ASIWPCA Report

The Association of State and Interstate Water Pollution Control Administrators (ASIWPCA) conducted a study at the request of the U.S. Environmental Protection Agency to assemble a baseline of information from state water quality agencies in 1984. The Association produced its report in 1985. It was the result of work performed by 49 states. Figures 8.36 and 8.37 summarize the principal findings of the sources and relative magnitudes of nonpoint source pollution. (J. WPCF, 1986)

Figure 8.36: Primary Nonpoint Sources in Impacted Waters



Source: After J. WPCF (July 1986)

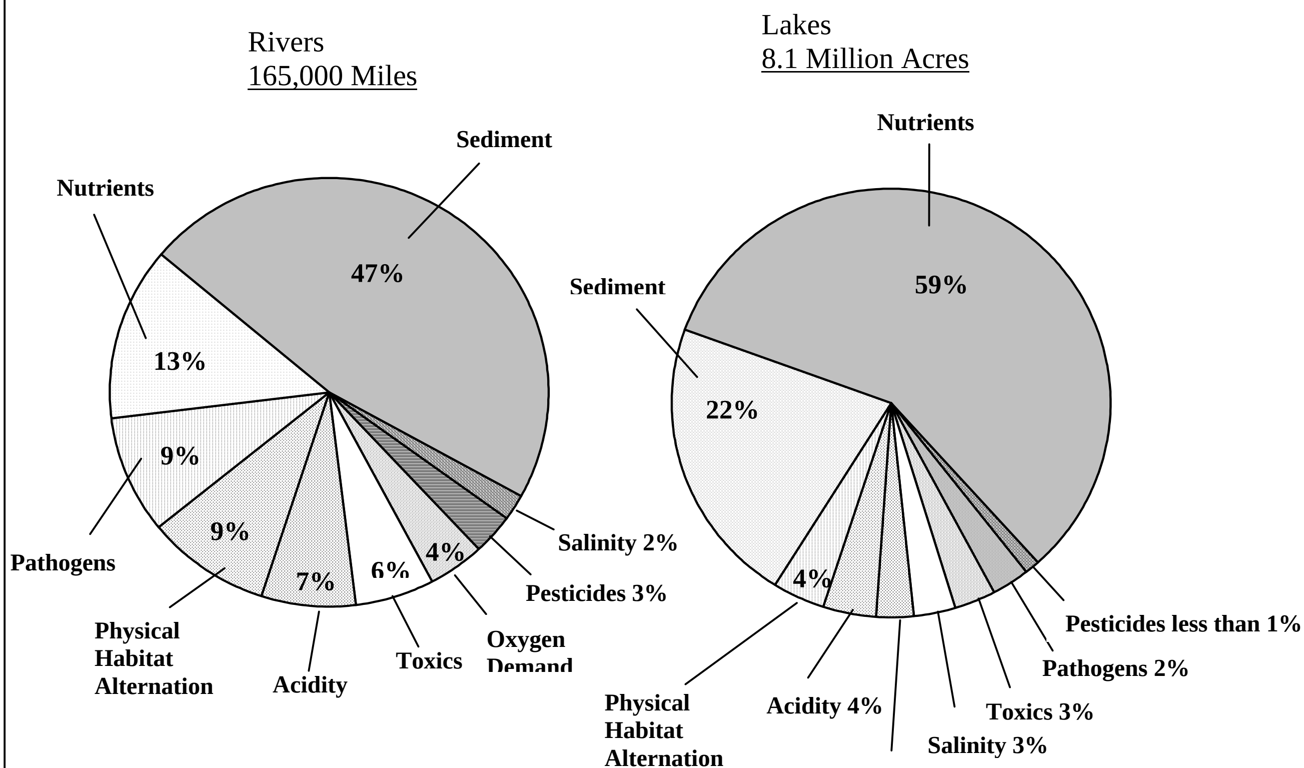
NURP Report

The U.S. Nationwide Urban Runoff Program (NURP) was conducted by the U.S. Environmental Protection Agency across the U.S.A. from 1978 for five years involving 28 projects. The data and the statistical methods used in data analysis were presented in a report. (U.S. Environmental Protection Agency, 1982). The final report contains the following major conclusions (U.S. Environmental Protection Agency, 1983(b)).

- Heavy metals (especially copper, lead and zinc) are by far the most prevalent priority constituents found in urban runoff.
- The organic priority pollutants were detected less frequently and at lower concentrations than the heavy metals.
- Coliform bacteria are present at high levels.
- Nutrients are generally present, but with a few site exceptions, concentrations do not appear to be high in comparison with other possible discharges to receiving water bodies.
- Oxygen demanding substances are present at concentrations approximating those in secondary treatment discharges.
- Total suspended solids concentrations are fairly high in comparison with treatment plant discharges.

· A summary characterization of urban runoff has been developed and is believed to be appropriate for use in estimating urban runoff pollutant discharges from sites where monitoring data are scant or lacking, at least for planning level purposes.

Figure 8.37: Primary Nonpoint Source Pollutants in Impacted Waters



Source: After J.WPCF (July 1986)

U.S. Federal Highway Administration Reports

The U.S. Federal Highway Administration (FHWA) has sponsored a number of studies relating to highway runoff water quality in the past 10 to 15 years. The following are the main highlights of some of the major reports published. Many of these reports were included in a literature review done for MTO (Dillon, M.M., 1990).

Kerri, K.D. et al, 1985

California highways produce pollutant loads in runoff water which are sufficiently low, so that costly treatment facilities are not needed to meet water quality objectives.

Dupuis et al, 1984

Monitoring of three sites with AADT of 7,400, 25,500 and 15,600 produces these conclusions :

- There are no apparent water quality impacts during storm events.
- Benthic invertebrate fauna population distribution, abundance and composition are not affected by runoff.
- There is no discernible impacts on periphyton communities.
- Undiluted highway runoff show no acute effects on organisms in bioassays. Some sub-lethal chronic effects were observed with undiluted runoff.

The report shows that runoff from highways with high AADT (185,000 and 50,000) has no toxic effects on aquatic biota (Dupuis and Kobriger, 1985).

Versar Inc., 1989

Versar Inc. concludes in a study that "Recent research indicates few significant impacts on receiving waters from highways with less than 30,000 ADT. There is little additional information directly relating highway runoff pollution to impacts on receiving waters. A common assumption in defining a highway runoff pollution problem is that data dealing with urban runoff effects are applicable in some degree." .

Stormwater Quality Management Design Criteria

In Ontario, stormwater quality design criteria are based on the requirements necessary for the protection of the quality of receiving waters. In 1991 the MOEE/MNR introduced the *Interim Stormwater Quality Control Guidelines for New Development*. Although the criteria provided in that document were specifically for new development, they have been used as a general guide to stormwater management in the province. These criteria provided water quantity control guidelines as a means for water quality control. The criteria required that runoff from 13 mm for warm water fishery, and 25 mm for cold water fishery, be detained for 24 hours. The criteria also encouraged infiltration, source control and highlighted that end-of-pipe facilities should be used as a means of last resort. However, these criteria were interim and they were expected to evolve as more end-of-pipe facilities came on line and performance monitoring became possible. It was expected that a better understanding of stormwater quality control was needed prior to proceeding further in developing new criteria.

In 1994 a new, more holistic approach was introduced by MOEE which promotes the adoption of stormwater planning on a watershed/subwatershed basis. This approach continues to be MOEE’s approach to date. It separates stormwater management practices into three levels of control: lot level controls, conveyance controls and end-of-pipe controls. This approach also promotes the design of end-of-pipe controls based on the modelling of suspended solids removal efficiency. This efficiency is used as an indicator of overall pollutant removal efficiency. The level of sediment removal efficiency is based on the *Fish Habitat Protection Guidelines for Developing Areas*, developed by MNR. These guidelines classify receivers into three levels of fish habitat protection, levels 1, 2 and 3 (in place of two levels of protection applied in the interim guidelines described above). A total suspended solids removal efficiency corresponds to each of the three levels of fish habitat protection levels. These removal efficiencies are 80%, 70%, and 60%, respectively. A fourth level of fish habitat protection is added by MOEE to account for situations of retrofit development. The corresponding suspended solids removal efficiency for this level of protection is 50%.

The new approach to stormwater management is described in MOEE's document *Stormwater Management Practices Planning and Design Manual*, June 1994. These criteria may be used as a general design guide of water quality control facilities. However, when developing a stormwater management system design for a highway project, the design criteria should be determined as part of the process for development of the project objectives and criteria. This should be done with the cooperation and involvement of environmental professionals, regulating agencies and the designer.

It should be noted that stormwater quality control is an evolving field and no firm policy in this regard is yet in place.

Stormwater Quality Computation Methods

What Methods to Select

The purpose of this section is to provide a description of the different approaches to the computations associated with the design of stormwater management works. Not all these approached are promoted by MTO nor are they all described in this manual. The preferred approaches will be identified. Two major choices need to be made successively:

1. Must the end-of-pipe stormwater management system accommodate the major and minor flows?

2. Which approach should be used in assessing pollutant removal efficiency, the empirical approach for contaminant buildup/washoff/removal or a critical particle size approach?

Without too much consideration of the working details of the methods at this stage, the following is a description of these two choices.

Drainage System

There are three possible scenarios for a drainage system.

Scenario 1 In this scenario, the drainage system is designed to convey the runoff of minor storms only (say, up to a 10-year storm). This scenario should be used only if the runoff from major storms, from direct and indirect drainage area, is properly provided for by a separate drainage system. If not, when a major storm occurs, the drainage system and the direct and indirect drainage areas will have flood problems. A drainage system that neglects the major drainage component in design is a faulty design.

Scenario 2 In this scenario, the drainage system is designed to convey the runoff of both major and minor storms. The runoff of all storms enters the BMP facility. The excess runoff which the facility cannot contain, overflows from inside the facility. If the facility is a pond, the solid contaminants previously settled in the pond may be scoured and resuspended. Some of the sediments may be washed out and pond performance is reduced. Proponents of this scenario assume that resuspension of sediments will not occur. This assumption may hold for minor storms, however, it should be verified based on pond design and performance for major and minor storms.

Scenario 3 In this scenario, the drainage system is designed to convey the runoff of both major and minor storms as scenario 2. Unlike scenario 2, however, the runoff is split into two streams at a point upstream of the BMP facility. One stream enters the BMP facility at a rate based on the design inflow rate of the BMP facility. The excess flow bypasses the BMP facility to the drainage system. Scenario 3 provides for stormwater quantity and quality management and takes advantage of the benefits of integrating stormwater quantity and quality management in planning and design. This scenario is the approach adopted in Chapter 3, Part 1 and Chapter 4, Part 2 of this manual.

Assessment of Pollutant Removal Efficiency

Empirical Approach This approach considers that contaminants go through three processes in sequence:

(1) Contaminants accumulate on surfaces of roads continually.

(2) Some or all the accumulated contaminants are washed off by runoff during a storm.

(3) The washed off contaminants go to a BMP facility through sewers and ditches. Some or all the contaminants are removed by the BMP facility. The unremoved fraction is discharged with the outflow.

In this approach, the mass of contaminants removed depends on the mass of dust and dirt accumulated.

The magnitudes of contaminants in (1), (2) and (3) can be estimated by empirical formulas or "representative" data taken from the literature or from direct data collection.

Estimation by Empirical Formulas Empirical formulas estimate dust and dirt accumulation by the length of road curb or by the area of a given land use. The formulas provided in the SWMM and the STORM models are typical examples. Researchers have shown that these formulas are deficient because atmospheric fallout contributes significantly to dust and dirt accumulation. They have also shown that atmospheric fallout and the contribution from scavenging are local phenomena and mainly dependent on wind velocity, frequency and direction; topography of the drainage area; total precipitation; and duration and frequency of temperature inversions. (Randall C.W., 1978; Shivalingaiah B. and James W., 1986).

Estimation by "Representative" Data The concept of using available data for making estimates in the above processes is recognized. It is important, however, that the quality of the data should be acceptable. The quality of stormwater quality data depends on two key factors: data sampling design and data analysis and reporting.

For stormwater quality data to be considered valid statistics, the data should satisfy certain principles of statistics, for example, sample size, sample timing and frequencies and so on. Adherence to statistical principles in data sampling design is very important because stormwater quality data depend on many parameters and these parameters vary widely. Extensive discussions in authoritative literature have made this point (Bodo B. and T.E. Unny, 1983; Herricks, E.E. et al., 1985; Yaksich S.M. and F. V. Verhoff, 1983).

Similarly, data analysis and reporting should satisfy certain statistical principles. The American Society of Civil Engineers has set this standard for authors of papers submitted for publication:

"If experimental data and/or relations fitted to measurements are presented, the uncertainty of the results must be stated. The uncertainty must include both systematic (bias) errors and imprecisions", (J. Environmental Engineering Division, Vol. 120(6), ASCE, NY, 1994.).

Finally, even for data of good quality, they are suitable for use in the project in hand only if the conditions of the original project and the project in hand are comparable.

It is difficult to find reports of stormwater quality projects and studies demonstrating that they satisfy the statistical principles mentioned above. The NURP reports are a notable exception (U.S. Environmental Protection Agency, 1982).

Critical Particle Size Approach This approach is prompted by the difficulty in getting reliable estimates for accumulation, washoff and particle size distribution of contaminants. It is a "guaranteed design" approach that aims at designing a pond to remove a critical particle size of contaminants specified for the project. This design can be achieved by using recognized formulas for settling of solid particles according to the law of physics (Adams B.J., 1995).

At a first glance, this approach may raise a concern that it does not provide an indication of the mass or percentage of contaminants removed. But there are two choices. If contaminant accumulation cannot be estimated with reasonable reliability, does is make sense to count on the result obtained for contaminant removal? If not, then the absence of this number should not be of concern. If the mass of contaminants removed is still required, it can be calculated using the specified critical particle size and the data of accumulation and particle size distribution of the contaminants. Refer to the design example in Chapter 4.

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Appendix 8A: Computer Models

Hydrologic Program Summary

Program Name:	**HSP-F** (Hydrological Simulation Program - Fortran), June, 1984.
Developer/ Distributor:	U.S. Environmental Protection Agency, Environmental Research Laboratory, Athens, Georgia.
Application: Runoff	Continuous simulation model for rural and urban land uses.
Simulation:	Sophisticated soil moisture accounting procedures that include storage algorithms for interflow, interception, upper and lower zones and groundwater. Hydrographs are generated using the kinematic wave approximation.
Routing Simulation:	Hydrologic routing procedures for open channel man-made or natural watercourses. Sewer flow, backwater effects and surcharging are not calculated.
Experience Requirements:	Significant hydrologic and modeling experience is required.
Time Requirements:	Large time requirements are required for learning and establishing data sets.
Data Requirements:	Large meteorological data sets required.
Computer Requirements:	Micro, mini and mainframe environments. Long computation times on microcomputers for moderate applications.
Comments:	Not normally used in the design and analysis of highway drainage systems.

Program Name:	**HYMO** (Hydrologic Model), 1973.
Developer/ Distributor:	J.R. Williams and R.W. Hann Jr., Agricultural Research Service, United States Department of Agriculture, May 1973.
Application:	Simulates runoff for single events from rural or undeveloped basins. Developed for planning and flood forecasting.
Runoff Simulation	Unit hydrograph with the SCS curve number procedure.
Routing Simulation:	Routing of hydrographs through natural open channel watercourses and through reservoirs. Uses the variable storage coefficient method.
Experience Requirements:	Some understanding of hydrologic processes.
Time Requirements:	Moderately low time requirements to learn and operate.
Data Requirements:	Information easily abstracted from AES, topographic maps and soil maps.
Computer Requirements:	Micro, mini and mainframe versions. Short computation times.
Comments:	Many hydrologic studies have been carried out in Ontario using HYMO. OTTHYMO contains all the HYMO subroutines and includes several to simulate urban conditions.

Program Name:	**ILLUDAS** (Illinois Urban Drainage Area Simulator)
Developer/ Distributor:	M.L. Terstriep and J.B. Stall. Distributed by Illinois State Water Survey.
Application:	Single event urban simulation model.
Runoff Simulation:	The program uses Horton's equation to calculate runoff and the Isochrone method to compute hydrographs from pervious and impervious basins.
Routing Simulation:	Hydrographs are routed through the sewer system using the storage indication method or a kinematic wave approximation. Reservoir routing not included. Surcharged sewer conditions not considered in detail.
Experience Requirements:	Some understanding of urban hydrologic processes.
Time Requirements:	Relatively low learning and application times.
Data Requirements:	Relatively low. Included are pipe sizes and slopes and general information obtained from topographic mapping.
Computer Requirements:	Micro, mini and mainframe environments. Small amounts of computation time required.
Comments:	ILLUDAS is an adaptation of the British Road Research Laboratory program. The designer/modeler should be aware of the following program limitations: (i) basin area less than 13 km^2 (5 mi^2); (ii) basin should not contain extensive agricultural or undeveloped areas; (iii) sewer and channel slopes should be less than 5%; (iv) simulated runoff should have return periods less than 10 years; (v) a minimum of 15 subbasins; and (vi) sewer and channel lengths less than 600 m (2000 ft.).

Program Name:	**MIDUSS** (Microcomputer Interactive Design of Urban Stormwater Systems).
Developer/ Distributor:	Alan A. Smith Incorporated.
Application:	Interactive, single event, design oriented program for urban and rural catchments.
Runoff Simulation:	Infiltration options are a runoff coefficient, a modified SCS curve number method, the moving curve Horton equ. and the Green-Ampt method. Overland routing options are a dynamic triangular response function, a dynamic rectangular response function, a single linear reservoir and the SWMM/RUNOFF overland flow algorithm.
Routing Simulation:	Routes hydrographs through circular sewers and channels of triangular, rectangular or trapezoidal cross-section using a modified Muskingum-Cunge method with internal adjustment of time step and reach-length for stability. Reservoir routing uses the storage indication method with a check on maximum allowable time step. Backwater and surcharge conditions are not considered.
Experience Requirements:	Some hydrologic understanding is required.
Time Requirements:	Relatively small amounts of time are required to learn and apply the model.
Data Requirements:	Included are pipe sizes and slopes and general information obtained from topographic mapping.
Computer Requirements:	Microcomputer environment.
Comments:	Program provides tables of suggested pipe sizes and grades for peak inflow and does part-full uniform flow analysis for pipe size and slope selected by the user. Detention pond sizes are suggested for user-specified peak attenuation.

Program Name:	**OTTHYMO-89** (Ottawa Hydrologic Modeling), 1989.
Developer	Paul Wisner & Associates Inc.
Distributor:	Greenland Engineering Group Corp.
Application:	Simulation of single events from urban and rural land uses. Developed for master drainage plans and stormwater management alternatives.
Runoff Simulation:	Runoff is simulated using several methods, namely, Horton's infiltration equation, the SCS curve number procedure, an improved CN* method developed at the University of Ottawa (initial abstraction is variable), and a proportional loss method. Hydrographs are created using unit hydrograph procedures or a kinematic wave approximation.
Routing Simulation:	Hydrographs are routed through sewers, culverts ot conduits, natural watercourses and reservoirs. Backwater, and surcharged sewer conditions are not considered.
Experience Requirements:	Understanding of hydrologic procedures.
Time Requirements:	Moderate time requirements for learning and application.
Data Requirements:	Information easily abstracted from AES, topographic maps and soils maps.
Computer Requirements:	The model can be used on IBM PC and compatibles
Comments:	Modified version of OTTHYMO-83. OTTHYMO-89 has the capability of generating two types of design storms from intensity-duration-frequency data and can incorporate a modified areal distribution factor.

Program Name:	**OTTSWMM** (Ottawa Stormwater Management Model)
Developer/ Distributor:	IMPSWMM Program, Department of Civil Engineering, University of Ottawa.
Application:	Analysis of urban drainage systems using single event storms. Models dual drainage system including street and sewer flow. Models the impact of inlet control devices.
Runoff Simulation:	Urban single event simulation model. Horton infiltration equation and kinematic wave hydrograph generation method. Models flow at catchbasins and inlets.
Routing Simulation:	Sewer routing by kinematic wave approximation. Surcharged flow and backwater can be calculated in the EXTRAN subroutine. Major and minor flow paths do not have to outlet at the same location.
Experience Requirements:	Understanding of hydrologic and hydraulic processes.
Time Requirements:	Dependent upon application, but generally high.
Data Requirements:	Dependent upon application, but generally high.
Computer Requirements:	Micro, mini or mainframe computers. Computation times are high for the EXTRAN subroutine.
Comments:	Only program to model separately flow in the streets and in sewers (dual drainage concept). Program is a modification of the U.S. EPA SWMM program. The OTTSWMM program has the following four (4) main subprograms: (i) surface runoff; (ii) inlet submodel; (iii) minor system submodel; and (iv) major system submodel.

Program Name:	**QUALHYMO** (Quality Hydrologic Model)
Developer/ Distributor:	IMPSWMM Program, Department of Civil Engineering, University of Ottawa.
Application:	Continuous urban and rural quality and quantity simulation program. QUALHYMO used for the planning analysis of new developments.
Runoff Simulation:	Runoff is calculated using a modified SCS Curve Number procedure. Hydrographs are generated using unit hydrograph procedures. Determines pollutographs.
Routing Simulation:	Kinematic wave approximation is used for channel and sewer routing. Quality routing through reservoirs is simulated.
Experience Requirements:	A good understanding of hydrologic and modeling procedures.
Time Requirements:	Moderate time requirements for learning and setting up program.
Data Requirements:	Long term precipitation records.
Computer Requirements:	Micro, mini or mainframe environments. A mainframe is recommended for large data sets.
Comments:	Developed and released within the last few years. Not normally used in the design and analysis of highway drainage facilities. Model has been calibrated and verified for Ontario conditions.

Program Name:	**STORM** (Storage Treatment Overflow Runoff Model), 1974.
Developer/ Distributor:	U.S. Army Corps of Engineers, Hydrologic Engineering Centre.
Application:	Continuous runoff and water quality simulation program. Developed to analyze storage requirements for treatment plants.
Runoff Simulation:	Hourly runoff is computed using SCS curve number of runoff coefficient methods. Hydrographs are generated using a Rational Method or a triangular unit hydrograph approach. Flood frequency curves developed from the output. Continuous accounting for soil conditions and six pollutants.
Routing Simulation:	No storage or channel routing is included in the program.
Experience Requirements:	A simple understanding of hydrologic and modeling procedures.
Time Requirements:	Moderate amounts of time are required to set up and apply the program.
Data Requirements:	Long term precipitation records are required for application.
Computer Requirements:	Microcomputer, mini or mainframe environment.
Comments:	No sewer or reservoir routing and only one subbasin limits the use of this program. STORM is not normally applied in the design and analysis of highway drainage systems.

Program Name:	**SWMM IV** (Storm Water Management Model), 1988.
Developer/ Distributor:	United States Environmental Protection Agency.
Application:	Single event and continuous urban simulation model. Analysis of complex sewer systems. Includes water quality simulation.
Runoff Simulation:	Horton's infiltration equation and kinematic wave routing for hydrographs. Includes snowmelt and continuous simulation.
Routing Simulation:	Sewer routing (peak flow rates) uses the kinematic wave approximation. Levels and flows for surcharged or backwater conditions can be calculated using the EXTRAN subrouting. EXTRAN uses time steps from five (5) to 15 seconds.
Experience Requirements:	Understanding of hydrologic and hydraulic processes.
Time Requirements:	Dependent upon application, but generally high.
Data Requirements:	Dependent upon application, but general high.
Computer Requirements:	Micro, mini and mainframe versions. Computations are time consuming for the EXTRAN subroutine.
Comments:	Several Ontario calibration and verification studies. SWMM is made up of five computation blocks; RUNOFF, TRANSPORT, EXTRAN, RECEIVE and STORAGE/TREATMENT. Calibration is strongly recommended for remedial measures. Surcharged water levels should be carefully reviewed.

Hydraulic Program Summary

Program Name	:	**DWOPER** (Dynamic Wave Operational Model)
Developer/ Distributor	:	U.S. National Weather Service.
Application	:	The program calculates water levels and flow rates for complex hydrodynamic flow conditions. Accounts for the effects of lateral inflows, reversing flow, locks, weirs and dams.
Routing Simulation	:	Models unsteady gradually varied flow conditions using the St. Venant, momentum and continuity equations.
Experience Requirements	:	The program requires users with significant experience and understanding of hydraulic processes.
Time Requirements	:	Significant time is required for learning, applying and analyzing program results.
Data Requirements	:	Extensive data required to operate the program. Included is main channel and storage geometry, hydrographs and roughness coefficients.
Computer Requirements	:	Micro, mini or mainframe computers. Large computation times for most applications.
Comments	:	DWOPER requires hydrographs as input. DWOPER is not normally used in the design and analysis of highway drainage systems.

Program Name	:	**EXTRAN** (Extended Transport)
Developer/ Distributor	:	U.S. Environmental Protection Agency.
Application	:	The program calculates flow rates and water levels in sewer systems with reversing flows, surcharging and backwater effects. Program has been used extensively to analyze large complex sewer systems.
Routing Simulation	:	The program models unsteady gradually varied flow conditions in sewer systems. It requires the physical characteristics of the sewer system, roughness coefficients and inflow hydrographs. Typical time steps range from 5 to 15 seconds.
Experience Requirements	:	A good understanding of hydraulic and modeling processes is required.
Time Requirements	:	Large time requirements to learn, set up and analyze the output results.
Data Requirements	:	Moderate amounts of data required to operate the program.
Computer Requirements	:	Micro, mini or mainframe environments. Large computation times as a result of short time increments.
Comments	:	EXTRAN is not normally used in the design of highway drainage systems.

Program Name	:	**HEC-2** (Hydrologic Engineering Centre Program)
Developer/ Distributor	:	U.S. Army Corps of Engineers, Hydrologic Engineering Centre.
Application	:	Calculates water surface profiles for open channel watercourses. Includes methods to account for losses at bridges, culverts, weirs and embankments.
Routing Simulation	:	Program does not route hydrographs between points of interest but calculates water levels based on channel geometry, flow rates and roughness coefficients. Program assumes steady, gradually varied flow conditions.
Experience Requirements	:	A good understanding of open channel hydraulic processes is required and some understanding of modeling procedures.
Time Requirements	:	Dependent upon application and experience.
Data Requirements	:	Large amounts of data for moderate hydraulic problems.
Computer Requirements	:	Program can be operated on micro, mini or mainframe computers.
Comments	:	Used extensively in the development of floodplain mapping.

Program Name	:	**WSPRO** (Water Surface Profile)
Developer/ Distributor	:	U.S. Geological Survey, Water Resources Division.
Application	:	Applied to the design and analysis of highway bridges and culverts. Calculates water surface profiles.
Routing Simulation	:	Calculates water levels at highway crossings for steady gradually varied flow conditions. Calculations account for submerged, weir and free surface flow conditions.
Experience Requirements	:	Moderate understanding of hydraulic processes.
Time Requirements	:	Moderate time requirements for learning, setting up and applying the program.
Data Requirements	:	The watercourse is described by a series of cross sections obtained through field surveys or from topographic mapping.
Computer Requirements	:	Program operates in micro, mini and mainframe environments.
Comments	:	Developed specifically for the design of highway bridges and culverts. The program was designed to improve existing backwater analysis programs such as HEC-2.

Program Name	:	**MOBED**
Developer/ Distributor	:	National Water Research Institute, Hydraulics Division.
Application	:	The program calculates water level, flow and sedimentation rates for dynamic flow conditions. Accounts for flow reversals in tidal areas.
Routing Simulation	:	Models dynamic flow conditions using one-dimensional St. Venant Equations. It has built in transport and frictional relations but these can be replaced.
Experience Requirements	:	A good understanding of hydraulic and modelling processes is required.
Time Requirements	:	Moderate to large time requirements to learn, set up and apply the program.
Data Requirements	:	Significant amount of data required.
Computer Requirements	:	Program operates on both micro and mainframe computers.
Comments	:	The model does not have a bridge simulation component.

Program Name	:	**HEC-15** - Hydraulic Engineering Circular No. 15, Version 2.1
Developer/ Distributor	:	D.L. Richards, Simons, Li and Associates Inc./U.S. Federal Highway Administration.
Application	:	Design of Roadside Channels with Flexible Linings.
Routing Simulation	:	Not Applicable.
Experience Requirements	:	A basic understanding of tractive force theory.
Time Requirements	:	Moderate time required for inexperienced designers.
Data Requirements	:	Discharge rates, lining material, slope, section shape and dimensions.
Computer Requirements	:	Microcomputer.
Comments	:	Program uses design procedures outlined in Hydraulic Engineering Circular No. 15, 1988, U.S. Federal Highway Administration.

Program Name	:	**FERNS** (Finite Element River Network Simulation)
Developer/ Distributor	:	Environment Canada, Water Planning and Management Branch.
Application	:	The program solves transient flow conditions in rigid bed rivers and tidal estuaries for open channel networks or multiple channel situations with and without off-channel storage. Also applicable where conventional steady state models cannot be used.
Routing Simulation	:	One dimensional unsteady flow routing model.
Experience Requirements	:	A good understanding of open channel hydraulic processes is required and some understanding of modelling procedures.
Time Requirements	:	Dependent upon application and experience.
Data Requirements	:	Fairly extensive data is required to operate the program.
Computer Requirements	:	Program runs on mainframe and micro computers. Graphics support on mainframe only.
Comments	:	Program is sensitive to large changes between element nodes. Streams with man-made structures like bridges, weirs, dams etc. can be analyzed.

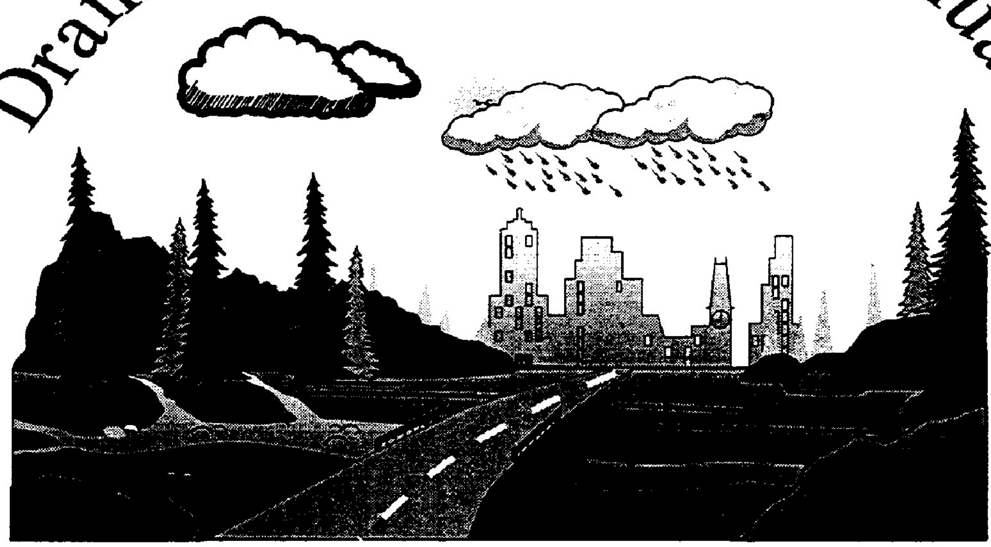
Program Name	:	**HEC-6** (Scour and Deposition in Rivers and Reservoirs)
Developer/ Distributor	:	U.S. Army Corps of Engineers, Hydrologic Engineering Centre.
Application	:	The program calculates water and sediment bed surface profiles. The change in bed elevation, water surface elevation and thalweg elevation are computed for each cross section along with the total sediment load with the efficiencies for clays, silts and sands.
Routing Simulation	:	One dimensional steady flow model. No provision for modelling development of meanders. Cross sections are divided into fixed and movable bed. The movable bed can either rise or fall. The one dimensional energy equation is solved using the standard step method.
Experience Requirements	:	A good understanding of open channel hydraulic processes is required and some understanding of modelling procedures.
Time Requirements	:	Dependent upon application and experience.
Data Requirements	:	Large amounts of data.
Computer Requirements	:	Program operates on microcomputers and mainframes.
Comments	:	Used in sedimentation studies in shallow reservoirs, downstream from dams or in natural rivers where both scour and deposition are involved.

Program Name	:	**Flow 1-D**
Developer/ Distributor	:	Environment Canada, Water Planning and Management Branch.
Application	:	The program determines transient flow conditions in rivers for divided or multiple flow channels.
Routing Simulation	:	Models flow conditions using the St. Venant equations including conservation of mass and momentum.
Experience Requirements	:	A good understanding of hydraulics and modelling processes is required.
Time Requirements	:	Large amount of time required to learn, set-up and apply the program.
Data Requirements	:	Significant amount of data required.
Computer Requirements	:	Program runs in micro environment with Fortran compiler and Graphics support.
Comments	:	Bridges have to be modeled separately as control nodes.

Ministry of Transportation

Ontario

MTO Drainage Management Manual



Chapter 9
Basic Stream Geomorphology for Highway Applications

Drainage and Hydrology Section
Transportation Engineering Branch
Quality and Standards Division

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General Discussion of Stream Geomorphology

Geomorphology of Streams

The field of geomorphology is a rapidly evolving science as noted by Chorley, Schumm and Sugden (1984), and involves study of a variety of earth processes such as fluvial, glacial and marine processes. Over the years there have been many publications on geomorphology and physical geology (Dury 1959; Hamblin, 1985; Selby, 1985; Garner, 1974) which provide an overall perspective on earth processes while other publications have pertained directly to fluvial processes (Leopold et al, 1964; Richards, 1982; Knighton, 1984; Dunne and Leopold, 1978). This manual cannot adequately treat the factors involved in establishing the occurrence and rates of all of the above earth processes, but will indicate an approach whereby the fluvial processes can be assessed resulting in a basis for sound design of water crossings that may be environmentally sensitive to fish and wildlife.

Fluvial geomorphology refers to the development of streams within deposits of the material being transported by the stream; that is, streams are self formed. The fluvial processes of erosion and deposition for an idealistic stream system are shown on the left side of Figure 9.1a* which indicates that sediment is eroded from the upper watershed, transported over relatively flat-sloped foot hills and finally deposited in deltaic zones. For Ontario, mountain ranges are absent and the idealistic scenario changes to small streams originating on Canadian Shield bedrock or glacial deposits as shown in Figure 9.1b. The streams flowing through glacial deposits frequently encounter lag boulder channel controls or stiff clay banks, that is, materials that are remnants of glacial ice processes. Large boulders may have been moved and deposited along ice margins by flowing water and clay deposits could have been the result of marine deposition during the recent ice age. Therefore, modern fluvial geomorphology processes in Ontario as well as other parts of Canada interact with older glacio-fluvial processes.

* This chapter uses many photographs and drawings to illustrate the text. For better continuity of the text, all the photographs and drawings are placed at the end of the chapter.

In this chapter, fluvial geomorphic processes will be treated starting with the presentation of basic fluvial principles occurring along streams and these are summarized as follows:

- If the supply of bed-material from the upper catchment is roughly equal to the bed-material transport capacity of the stream channel, then the stream is stable.
- If the supply of bed-material is much less than the bed-material transport capacity, then the stream bed will lower (degradation).
- If the supply of bed-material is higher than the bed-material transport capacity, then the stream bed will rise (aggradation). Often, aggradation results in creation of bars and islands which cause deflection of flow currents leading to bank erosion.
- The plan form of a stream is related to the flow in the stream, the slope of the landscape and the capability of the stream to transport sediment.

While the above principles appear rather simple, the recognition of the ongoing fluvial process in the field and from maps and air photos can be difficult. Therefore, this manual proceeds through distinct design steps leading to an assessment of the fluvial process as well as to measures contributing to improved fish habitat. The fluvial processes terminology is defined in the glossary.

The dominant variables determining the stream channel size and shape are water discharge (Q), bed-material load (Q_s) and valley slope. Valley slope is imposed and strongly influences the pattern of the developed channel which can result in a channel slope being flatter than the valley slope. The adjustment of a stream channel can occur at several different time scales (Figure 9.2):

X bed configuration whereby bed forms can develop from ripples to dunes to bars and take a short time (days) to undergo changes;
X cross section shape, width and depth, which can change over a flood period of days or months;
X channel pattern such as meandering which takes one flood season or many average flow years to change; and
X channel bed slope which generally takes decades to alter significantly under natural processes, but can change faster if major changes, such as lowering of base level, are imposed.

However, prior to dealing with the previously mentioned channel changes the next section will initially deal with the setting of the stream channel and then will present the fluvial principles which establish the stability of the stream channel. If the stream is assessed to be unstable, changes to bed slope will have to be estimated. Several estimating techniques will therefore be presented so that the designer can assess how fast a slope may change.

Water Crossings and Geomorphology

The most frequent interference experienced by streams would be bridges or culverts. The introduction of a water crossing can influence flow levels and velocities which may induce changes to the channel geometry and the stream bed profile. A study of bridge failures by Smith (1977) indicates that out of 143 failures, 70 (49 %) are caused by floods and foundation movement (scour and/or degradation). The understanding of fluvial processes such as aggradation, degradation and scour are imperative if bridge failures due to fluvial processes are to be reduced. Also, the recent awareness of the need to replace or upgrade bridges (U.S. Dept. Transportation, 1991) provides a new opportunity for engineers to re-assess environmental conditions around bridge sites and to build in measures to improve fish habitat. For sustained fish production, a stream should have stream side vegetation cover, food for fish, clean gravel beds, a sequence of pools and riffles, clean water and adequate flows. Bridge crossings should be so designed that these characteristics are not lost.

As an example of the complexity of the interaction of infrastructure along a transport corridor with fluvial processes down a valley, consider the case shown in Figure 9.3. A new highway has been located along a stream valley which also includes several powerlines, a railway and local borrow pits. At several locations, the river bank has to be protected to prevent lateral shifting of the stream into the highway (at A and B) while scour protection is also required at the foundations of several powerline towers (C and D). The design of the highway bridge at E involves assessment of bank erosion, potential lateral shifting of the channel, and potential vertical changes of the stream bed. At the upstream end of this reach, at F, the stream bed is lined with boulders and appears stable vertically, while further downstream the presence of mid-channel bars and large point bars indicates aggradation and an unstable channel to G where the stream appears to enter a confined gorge. Terrace erosion at H will also supply sediment to the unstable reach adjacent to the bridge crossing.

The highway has been located along the lowest part of the flood plain while the railway is at a higher elevation on the left terrace. It may have been better to locate the highway crossing at K where the approach condition would be perpendicular to the bridge axis and lateral shifting would be minimal.

This stream shows a pool and riffle sequence and shaded reaches which would serve as good fish habitat zones and the bridge crossing at E has not disrupted the stream processes. These conclusions can only be arrived at from a geomorphology design which is the primary aim of this chapter.

The above example shows potential problems related to bank erosion and lateral shifting which may endanger infrastructure, but may not be detrimental to fish habitat. Another common water crossing problem is that of degradation (bed lowering) with the most probable cause being a stoppage of sediment supply from upstream such as when a dam is constructed or gravel excavated from a stream. An example is shown in Figure 9.4 where gravel excavation has caused upstream progressing degradation and undermining of bridge piers. The degradation process generally results in removal of fine material from the stream bed leaving large cobbles and boulders which may be detrimental to fish, especially where a series of pools and riffles is transformed into one long, turbulent riffle.

Another example of the value of a geomorphology component in bridge crossing design is the case of stream bed aggradation on a delta (Figure 9.5). A railway bridge crosses a stream channel on an active delta and the progressive deposition has resulted in no clearance to pass flood flows. Because of recent construction of a golf course on each side of the stream and because of environmental restrictions, the material excavated from the channel has to be transported to a storage site several kilometres from the bridge. Annual maintenance is costly. A geomorphic analysis could have resulted in a higher level for the railway bridge deck or for a stream diversion which would bypass the majority of sediment into the lake somewhere upstream while still retaining a low flow channel through the deltaic reach. The geomorphic analysis should be followed by an environmental assessment of potential impact of sediment deposition on the delta on fish and wildlife.

Water Crossings and Fish Habitat

The *Environmental Manual: Fisheries (Working Draft),* Ontario Ministry of Transportation (1994) shows a number of hydraulic features important for fish habitat and these are as follows:

- low gradient riffles;
- rapids;
- cascades;
- glides;
- backwater pool associated with rootwads;
- backwater pool associated with boulders;
- secondary channel pool; and
- dammed pool associated with large debris.

While the features are desirable, there is no optimum combination of these features for a specific stream - generally combinations of pools, secondary channels and riffles results in a stream reach that is utilized by a variety of fish species. Turbulent shallow water along gravel riffles increases oxygenation of the water and provides conditions for spawning while deep pools have low velocities and are rich in organic matter. The two different types of stream features provide the different requirements of fish at different stages of their life cycle. Pools and riffles are common to meandering channels which also exhibit steep eroding banks, sheltered (treed) reaches and open point bars. As stated by HRL (1988) - "Meander removal immediately reduced the structural diversity of a river and as a consequence its biological richness. Also, less drastic action to ease meanders can be equally serious as the complexity of bank slopes, channel depths and sediment distribution will be lost!"

The early assessment of a water crossing should identify the above mentioned features and indicate whether a crossing will modify the more desirable features for fish habitat. An example of the loss of certain hydraulic features is shown in Figure 9.6 where an eroding bend having deep pools and logs has been transformed to a riprapped bank and no pools.

There are usually several options available for a water crossing and these options can be related to:

- types of bridges and culverts;
- modifications to the streams; and
- quality of fish habitat and compensation necessary due to modification of the existing stream.

Therefore, the options can be many. In order to simplify this introductory discussion, we will consider that the water crossing will not adversely constrict the active river cross section, then the options could be reduced to the modifications at the crossing and the necessary habitat compensation.

The following stream modifications (Table 9.1) are typical:

- armoured banks;
- constrictions;
- cutoffs; and
- channelization.

Table 9.1: Stream Modification Measures

Change to Habitat	Stream Modification Measures			
	Armoured Banks	**Constriction**	**Cutoff**	**Channelization**
Vegetation.	X Probably no change to trees. X Possible impacts to aquatic and riparian vegetation.	X Some trees removed. X Possible impacts to aquatic and riparian vegetation.	X No change to trees. X Possible impacts to aquatic and riparian vegetation.	Extensive removal of trees, and aquatic and riparian vegetation.
Change to bed material.	Bed armoured stones and boulders.	Bed degrades, becomes coarser.	Bed degrades, becomes coarser.	Bed degrades, becomes coarser.
Lateral changes.	More stable.	Higher velocities, banks require armouring.	New erosion of banks.	More stable.
Bed area available to fish.	Less because bed becomes coarser.	Reduced.	Reduced.	Reduced substantially.
Quality/quantity of habitat	X Potential loss of pools and riffles. X Undercutting of banks.			

Most of the modifications result in reduced pools and riffles, less trees and sometimes much less bed area available for spawning and rearing. Therefore compensation measures become essential.

The stream modifications have historically not considered the changes to fish habitat and have not been based on geomorphic patterns often taken on by natural river channels. Frequently, the channelization has resulted in changing a meandering system with overhanging trees to a straight-line, riprapped channel having no pools or riffles. This chapter will provide principles for the creation of a pool-riffle sequence down a relatively straight channel as shown in Figure 9.7. Therefore, prior to setting out habitat compensation, the consideration of geomorphic principles may lead to a modification at the water crossing that is compatible to fish. Also, deep scour pools can be developed by placing of obstructions which again could comprise desirable habitat.

This chapter also sets down ten (10) steps leading to assessment of stream stability which includes consideration of fish habitat and accounts for the above possible geomorphic changes over the short as well as long term.

Assessment of Stream Stability

Introduction

The assessment of stream stability can proceed in an orderly manner beginning with broad-scope appraisal of the setting of the stream along the reach being studied, then to analysis of fluvial processes and their rates with consideration of environmental aspects. The procedure for stream stability assessment is set out in Figure 9.8 and outlined below.

Ten Steps for Assessing Stream Stability

The following design steps are suggested:

1. Assess the type of geomorphic setting in the vicinity of the crossing.
2. Evaluate environmental conditions, especially fish habitat and present methods for protection.
3. Evaluate fluvial (water-sediment) processes.
4. Assess the characteristics of the stable channel; checking width, depth slope, and channel pattern of the stream. If the channel is stable, go to Step 10. If the channel appears to be degrading, go to Step 5. If the channel is aggrading, go to Step 6.
5. Assess degradation process.
6. Assess aggradation process.
7. Assess bank erosion - lateral shifting processes.
8. Assess rates of stream channel changes.
9. Reiterate the degradation and aggradation assessment to better understand these processes in three dimensions.
10. Finalize conclusions for planners and designers.

Step 1: Assess Geomorphic Setting

Streams generally exhibit a specific pattern and channel shape over long lengths and these characteristics are frequently influenced by the landforms through which the stream flows which, in turn, can be controlled by geologic features. Through some stream reaches the landform may be created by water and sediment depositing adjacent to the stream - in other words, the stream and landform are changing with time (dynamic). In other geomorphic settings, the stream character is controlled by rock outcrops and the boundaries can be considered as permanent. A variety of channel types is shown in Figure 9.9 from Mollard and Janes (1984) and the following general geomorphic settings are to be found in Ontario and are summarized in Figure 9.10:

- bedrock or boulder bed valleys;
- intermediate width, partly confined valleys;
- wide alluvial bed valleys;
- alluvial fan;
- flood plain or outwash plain; and
- delta.

This classification of geomorphic settings is also similar to a classification of river processes developed by Kellerhals, et al. (1976) whose works emphasize the fluvial features of a stream.

Bedrock or Boulder-bed Valleys

An example of a rather narrow bedrock valley with abrupt water falls is shown in Figure 9.11 where the width of the stream does not change because of the bedrock walls and the water-surface slope is totally controlled by bedrock and large boulder outcrops - the slope is not established by fluvial processes. The processes whereby the stream developed a gorge is part of the glacial history of the region as well as the history of the development of the geologic formations. A summary of the natural landscapes of Canada and their development is found in Bird (1972).

Another example of boundary control of a stream is boulder-bed valleys where large boulders line the entire stream bed for a relatively long reach as shown in Figure 9.12. Frequent boulder rapids control the stream longitudinal profile and the shape of the channel as shown in Figure 9.13. The hydraulic properties such as width, depth, slope and velocity are generally a function of the flow. They do not change with time since bedrock erodes slowly and large groupings of boulders do not become eroded during high flows. Catastrophic floods are required to mobilize boulder rapids and for this reason, these stream reaches are frequently utilized for crossings by highways, railways, pipelines, etc.

In Ontario, the Canadian Shield (also called the Precambrian Shield) predominates as shown in Figure 9.14. The Shield is characterized by frequent bedrock outcrops and is overlain by loose overburden due to glaciation (Mollard and Janes, 1984). As an example, the Albany River is typical of many rivers that flow across the Shield - the channel width varies along its length, it connects lakes and has frequent rapids as shown in Figure 9.15. The river flows through a moraine and is exceptionally wide at A where sediment has dropped out to form bars and narrow at B where the river has cut down to bedrock. The pattern is irregular with abrupt changes in direction. These Shield rivers are also characterized by frequent rapids and waterfalls such as shown in Figure 9.16 for the Burntwood River in Manitoba.

Intermediate Width, Partly Confined Valleys

Along a portion of a stream, the channel or channels flow through a valley whose width confines the free movement of the channel laterally and the subsequent channel pattern is partially controlled by resistant valley walls. Streams through this reach often have regular meandering channels such as shown in Figure 9.17 where the amplitude of the meander pattern is smaller than for wide alluvial plains because of confinement. The pattern has a dominant movement in a downvalley direction and this factor should be acknowledged in choosing a location for a bridge crossing. Figure 9.18 shows downvalley movement of a constrained meander. Bridges should span the entire width along the more stable portion of a shifting meander pattern. An example of a stream impinging onto a valley wall and causing instability is shown in Figure 9.19, for the Nottawasaga River, Ontario. The channel pattern is influenced by the slumping bank which deflects currents to the other side of the stream causing erosion along the right bank and subsequent changes to pattern.

Intermediate width valleys frequently exhibit streams that are partially entrenched into a historical fluvial or glacial deposit and have beds that are partially armoured so that only large flows can mobilize the material on the bed.

Wide, Alluvial-bed Valleys

This geomorphic setting frequently has a filled valley bottom, primarily with sands and gravels, and poses few restrictions to the movement of a stream. Within certain regions, valleys have a complex history and various features such as terraces and alluvial fans are common. Figure 9.20 shows a complex valley with a broad alluvial flood plain.

Alluvial Fan

Alluvial fans are deposits of primarily coarse sediments, ranging from boulders to sand. The central point of the fan, referred to as the apex, usually lies at the mouth of a canyon or gorge and the fan radiates over an alluvial plain or valley bottom. The stream channel over a fan is highly unstable because of deposition of bed material, and the most coarse material drops out near the apex. Many alluvial fans have been partly confined by dikes and the deposition process occurs over a confined space as shown in Figure 9.21.

Flood Plain or Outwash Plain

Flood plains are similar geomorphic features to wide alluvial valleys but generally do not have adjacent landforms such as alluvial fans. In many instances in Canada, flood plains have been former lake beds and have streams carved into lacustrine materials. These channels are characterized by rather slow lateral shifting, sinuous single channels and the Grand River in Southern Ontario (Figure 9.22) is a good example.

The Grand River, between Paris and Brantford, breaks through the Paris and Galt Moraines. The material through which the stream flows varies from stiff clays (near Lake Erie) to sands and gravels near the moraines (Figure 9.23). At Brantford, the stream slopes flatten from about 1:600 to about 1:2,500 (Chapman and Putnam, 1984). This is where the stream flows over silts and clays of the ancient Lake Warren. For many streams in southern Ontario, the construction of low dams or weirs has interfered with the fluvial processes by trapping sand and gravel resulting in some lowering of downstream beds. At Brantford, there are several weirs, one of which has now been removed, which have controlled flow levels and sediment transport for years and there are now four bridge crossings within a short distance as shown in Figure 9.24. The width of the river is also partly controlled by the old weir and the four bridges.

Rivers on stiff materials such as clays and silts generally take on a more tortuous meander pattern such as shown in Figure 9.25 for the Goulais River in Ontario. These rivers have flat water slopes and change their pattern slowly.

Delta

Deltas are landforms created by deposition of sediment in relatively tranquil bodies of water. Deltas are common to inland lakes where several major distributary channels exist. Large deltas in Canada are the Red and Saskatchewan River deltas in Manitoba and the Peace-Athabasca Rivers delta in Alberta. In Ontario, deltas are generally smaller as shown by the Goulais River delta in Lake Superior (Figure 9.26). The delta takes on a classical birds-foot delta pattern and is slowly building into the bay. Stream channels are usually slowly aggrading and may also be shifting laterally. After large floods, however, channels can also be abandoned or can suddenly take on larger flows.

Geomorphic Settings Common to Ontario

As discussed by Bird (1972), and shown in Figure 9.14 most of Ontario consists of the Canadian Shield with thin veneers of glacial till and a relatively small zone, referred to as the St. Lawrence Lowlands, which is typified by sand and clay plains with surficial features such as moraines and drumlins (Figure 9.23). Figure 9.10 shows how the various geomorphic settings change in a downstream direction which implies that classification systems for stream channels should segregate a river into specific reaches. Generally, geomorphic settings portray distinct dynamic fluvial processes such as degradation or deposition. Kellerhals, Church and Bray (1976) outline a procedure for classifying and analysing river processes which utilizes a codification of patterns, islands and bars to distinguish river processes. Recently, Rosgen (1985) has developed a "geomorphological classification". For details refer to *Natural Channel Systems, An Approach to Management and Design*, Ontario Ministry of Natural Resources (1994).

For some additional sources of information, refer to: Annable, W.K., 1994, Leopold, L.B., 1994 and Rosgen, D.L., in the references.

Step 2: Evaluate Environmental Conditions

Introduction

This section summarizes the requirements for fish habitat which are defined by fish biologists as part of the water crossing design team. In order to team up with biologists for this task successfully, engineers should be aware of the quality of existing habitat and ensure that this habitat is not destroyed by adverse geomorphic changes.

Much of the information contained in this section is based on the *Environmental Manual: Fisheries (Working Draft)*, Ontario Ministry of Transportation (1994). That manual provides a standard set of practices for fisheries data collections and assessments for each stage of the planning and design process, as outlined in the *Fisheries Protocol* (1993), an agreement between MTO and Ontario Ministry of Natural Resources (MNR). The goal of this protocol is to address the protection of fisheries resources during all stages of provincial highway undertakings.

Fish Habitat Requirements

Fish generally require the following habitat features for survival.

Stable Stream Flow Stream flow must be sufficient to prevent freezing or exposure of newly hatched fish and must provide a source of clean, well oxygenated water for respiration and the removal of metabolic wastes.

Clean Gravel Substrate The habitat requirements may vary for different species. For instance, juvenile fish, benthic organisms and aquatic insects require clean, stable gravel in which to hatch and rear. The deposition of silt or coarser sediment can smother developing eggs and embryos and can also lead to cementation of gravels, thereby, preventing deposition of eggs or use of gravels as habitat for aquatic invertebrates.

Riparian Zones The wet soil areas next to streams, lakes, estuaries and wetlands are called riparian zones. These areas have high water tables and saturated soils and are vegetated with moisture-loving plants and trees such as pine, birch, alder, willow, fir, and spruce. Riparian zones can be associated with several types of stream side cover, including overhanging vegetation, undercut banks, logs, rubble substrates, turbulence, and/or deep pools. These areas provide productive fish habitat, as well as protective cover for many aquatic organisms. Riparian vegetation also helps to stabilize stream banks and prevent erosion. Figure 9.27 indicates the riparian and fisheries sensitive zones in and around a typical watercourse.

Access Spawning and nursery areas must be accessible to adult fish and juveniles seeking rearing habitat.

Riffles and Pools A high proportion of riffles and pools is required for the successful production and maintenance of a diverse aquatic community. Typically pool and riffle spacing is shown in Figure 9.28.

Wetland Habitats Many aquatic species, including fish, depend on wetland habitats for food and shelter. These areas serve to retain storm water, prevent flooding, and improve water quality and also represent a source of groundwater recharge. Examples of wetland habitats include: swamps, bogs, fens, marshes, sloughs, potholes, wet meadows, river overflows, estuarine areas, tidal overflows, mudflats, shallow lakes and ponds. Wetland habitats also provide essential spawning and rearing areas for some species.

Evaluation of Fisheries Habitat

The *Environmental Manual: Fisheries (Working Draft*) provides a series of exhibits including a generalized decision framework for the evaluation of water body sensitivity to development activity, a model for ranking fish community and habitat function and a model for evaluating fish habitat values. As a first step, it is necessary to identify the function of the habitat (i.e., spawning, rearing, resting, feeding, and/or provision of base flow and associated food (energy) transfer).

Ideally, the identification of fish habitat, evaluation of fisheries resources, and assessment of potential impacts related to the installation and maintenance of steam crossings should begin early in the highway development process. To ensure that adequately informed decisions can be made as the project progresses from planning to design, highways staff and consultants must have access to fisheries habitat and resource information of relevance to the proposed project area. A qualified fisheries biologist is needed to evaluate aquatic resources and habitat, and to provide input into environmental design features and potential mitigation measures.

Knowledge of aquatic habitat characteristics in the vicinity of a proposed crossing is of critical importance, both to the evaluation of potential effects of water crossing construction, and ultimately, to bridge or culvert design. In addition to aquatic habitat requirements, consideration should be given to the physical attributes of the water body, such as stream order, stream gradient, substrate characteristics, cover types, geomorphology and water depths.

In some cases, due to lack of information regarding fish communities or habitat characteristics near a proposed crossing, it may be necessary to conduct field studies. Observations of the following parameters are of key importance in the evaluation of crossing-related impacts:

- substrate characteristics;
- estimated width and depth of stream and/or littoral zone;
- adjacent land use;
- presence and extent of riparian cover;
- morphological characteristics of the channel (pools, riffles, runs, side channels, backwater sloughs, etc.);
- bank stability;
- presence of significant seepage;
- incidental occurrence of fish or wildlife; and,
- surface water and air temperatures.

In a situation in which one or a combination of crossings may alter stream hydrology, it may also be necessary to collect detailed information regarding channel geomorphology and stream flow/velocity.

Fish Habitat Improvement

The *Environmental Manual: Fisheries (Working Draft)* and the *Fisheries Protocol* guide MTO staff and/or consultants on specific mitigation measures for design and construction of all water crossing structures. In addition to protecting fish and fish habitat, these measures may include provision for fish habitat improvement. Before undertaking any design or construction of fish habitat improvement or enhancement devices or structures, it is essential that project staff consult and obtain advice from DFO/MNR offices.

The main purpose of stream habitat improvements is to minimize the effect of physical limitations to fish populations created during water crossing construction. Examples of such limitations include loss of important stream features (e.g., riffle/run habitat, deep pools, spawning gravels, riparian vegetation, large organic debris) and decrease in stream habitat complexity. A pool and riffle spacing is shown in Figure 9.28.

The restoration and enhancement of fish habitat can be accomplished through various methods, such as adjustment of a stream's long profile to concentrate or dissipate flood flow energies;

addition or removal of pools and riffles; and, adjustment of gradient control points. Flow depth, velocity, continuity, hydraulic habitat structures, aeration, fish passage, and bed stability can be modified by analyzing and adjusting the channel profile and geometry without loss of flood capacity.

A variety of different structures can be used to improve or restore fisheries habitat. Selection of the most appropriate structure for habitat restoration, mitigation or improvement depends largely on site specific hydraulic conditions and the fish species of concern. Beside hydraulic and geomorphic considerations, the *Environmental Manual: Fisheries (Working Draft)* should be referenced to determine suitable instream structures. Figure 9.29 illustrates some stream enhancement techniques and devices.

It should be noted that streams with high velocities should not have structures placed in the main flow channel since these could be damaged. For this situation, fish habitat improvement should be confined to back channels or tributaries.

Refer to the *Environmental Manual: Fisheries (Working Draft)* which identifies typical devices and structures used for fish habitat improvement projects.

Step 3: Evaluate Fluvial Processes along Stream Reach

In order to understand the geomorphic history of a stream reach and to assess rates of fluvial change, a classification of fluvial processes has been suggested by various researchers such as Schumm (1977) whose fluvial system is shown in Figure 9.30. This system presented by Schumm has been modified by Galay (1987) by expanding Zone 2, the transfer zone, into a deposition (alluvial fan) reach and a transport reach as well as expanding Zone 3, the deposition zone into a deposition and deltaic zone. The argument for the modification is reportedly that a deposition zone incorporating a single channel would be distinctly different from a deltaic zone with many distributary channels. The idealized fluvial system is shown in Figure 9.31.

Throughout the idealized profile, the fluvial processes can be divided into four distinct categories as illustrated in Figure 9.31.

1. Erosion reach which is in the upper portion of a watershed where sediment supply originates;
2. Transport reach has few tributaries and is generally stable in terms of cross section shape;
3. Deposition reach has bars and sometimes multiple channels; and
4. Delta reach where most of the fine material drops out and the channels split into distributary channels.

Generally, the erosion reach is undergoing surface erosion and mass wasting (landslides) and the amount of material supplied to the stream channels is usually less than the sediment transport capability of the channel. In this reach, sediment transport equations should not be used to determine the amount of bed-material being moved during a specified time period, such as daily or monthly volumes, because the equations will yield volumes that would be larger than the actual rates. In other words, the supply of sediment is generally less than the capability of the stream to move the sediment.

Along the transport reach, the amount of sediment supplied to the reach is generally equal to the amount leaving the reach resulting in a stable channel. Bars may be in evidence, but the reach may be described as being in equilibrium and sediment transport equations can be used to arrive at estimates of bed-material load. This reach can be analysed and compared with "regime" equations that have been developed by engineers such as Lacey (1929) and Blench (1957, 1969). These regime reaches will be dealt with further in the next section.

Further down the stream system, as the water surface slope flattens, the coarser bed-material drops out, bars and islands are formed because the sediment transport capability is diminishing in a downstream direction and sediment is going into storage. This deposition reach generally precedes the delta reach where all of the fine material is dropping out along distributary channels which form a triangle-shaped delta.

The above idealized fluvial process reaches are usually more complex in nature and are sometimes interchanged, for example where an alluvial fan (deposition reach) follows directly from an erosion reach located along a gorge passing through a mountain range as shown in Figure 9.31.

A technique for classifying the fluvial processes is to plot a longitudinal profile of a reasonable length of river, say 30 km, near the bridge or culvert crossing.

A longitudinal profile of a stream, which extends upstream and downstream from the river crossing reach, can provide valuable information on the stability of a stream. The profile can be developed from field surveys or from topographical maps and should include tops of river banks (bankfull stage) if this data is available. From assessment of the profile, the following general conclusions can be made:

- the upstream end of the profile where the slope is steep is generally eroding;
- the downstream end of the profile which is rather flat with slopes generally less than 0.001 can be a stable stream reach where the capability for sediment transport is matched by the supply of sediment;
- in between the above two slopes there exists a concave upwards transition which is generally a deposition reach and frequently has a braided channel pattern; and
- the extreme flat slope where the stream approaches a base level (sea) can be referred to as a deltaic reach where most of the fine wash load (silts, clays) are deposited.

From a longitudinal profile one can infer which reaches of the stream are relatively stable and where water crossings may require additional river works to attain stable approaches to the crossing. An example of a profile is shown in Figure 9.32 and photographs of the deposition reach and the stable reach are shown in Figure 9.33.

Step 4: Assess the Characteristics of the Stable Channel

Once it is established that the stream is fluvial then the following check should be made. A quick comparison should be made of the hydraulic characteristics (width, depth and slope) of the reach in question to the characteristics of stable fluvial channels. Researchers have developed relationships for the hydraulic characteristics of width (b), depth (d), and slope (s) being dependent on water discharge (Q) which is generally taken as the bankfull discharge for a stream reach (Lacey, 1929; Blench, 1957; Leopold and Maddock, 1953; Simons Li and Assoc. 1982; and Neill, 1973).

The researchers used stable canals that generally did not change their characteristics over a long time period and these canals were termed "regime canals". The regime concept was extended to include rivers which added one further degree of freedom, namely, the pattern of the river or stream channel. The usefulness of comparing the studied stream reach to a regime channel is that one can quickly assess whether the stream will change significantly in the future.

If the stream being studied for a potential river crossing has a single channel with occasional islands and a sand bed, it can be checked for stability by plotting its width, depth and slope on the charts developed by Neill (1973) and shown in Figures 9.34a, b, c and d. If the values for the stream in question plot close to the plotted curves, then the stream can be considered relatively stable and would not be expected to change its hydraulic characteristics unless there is a major change in flow or in supply of sediment. If the slope is higher than the regime value, then the stream is likely transporting a high amount of bed-material and is relatively unstable.

However, it is important to note that regime equations apply only to a transport reach and that as one proceeds downstream into a deposition reach, the channel characteristics change with distance because bankfull discharge reduces due to spillage of flows onto the flood plain. In other words, the channel becomes smaller as one proceeds through a deposition reach and subsequently breaks up into several distributaries at the start of a delta.

Another step in assessing stability is to check meander properties of meander wavelength versus bankfull discharge as shown in Figure 9.34d. Unconfined meandering streams display a regular sinuous pattern. Also, stable streams may have tortuous or contorted meanders but not plot close to the plotted line in Figure 9.34d, and some of these patterns are shown in Figure 9.10. The meander characteristics and relationships for stable channels are shown in Figure 9.35. Also, if slopes are rather steep, then one should check the type of channel pattern to be expected for the reach by using Figure 13.36 which is from Leopold et al. (1964).

If the reach in question proves to be stable, then it will not be necessary to check for degradation and aggradation processes - one may then proceed to compute flow depths as presented later. The stable channel regime plots can also be used for design of diversion channels or flood channels.

The development of pools and riffles is characteristic of both meandering and straight channels flowing in coarse sands and gravels and they are known to have distinctive flow geometries. As shown in Figure 9.37, the spacing of successive pools is usually 5 to 7 times the channel width for straight reaches and probably larger spacing for a fully developed meander system. For fish habitat systems where compensation may be required, it may prove worthwhile to design a pool-riffle sequence that could serve as adequate habitat thereby reducing the necessity of **additional** compensation.

The use of the stable channel concept to transform unstable rivers into stable ones is standard practice in river engineering, but the technique has to be applied with caution and should acknowledge existing and future fluvial processes. Consider a relatively recent channelization of the East Fork San Juan River in Colorado (as reported by Dorward, 1990 and Rosgen, 1988). The San Juan's East Fork consisted of a highly braided system having a rather steep slope of 0.013 and a cobble-gravel bed having a median grain size of about 20 mm. The pattern is shown in Figure 9.38 and could be classified as a deposition reach. As explained by Dorward, (1990) - "developers have proposed building a major downhill ski area on National Forest and private land, with a four-season resort at its base ... a plan was developed to reconstruct almost a mile (1.6 km) of the stream, confining it to a new meandering channel of the same capacity with stabilized banks". The re-constructed meandering system is shown in Figure 9.38, from Rosgen (1988) who further reports that "a doubling of the shear stress value from the braided channel to the new C1 stream type without change in stream slope. . . allows the new channel to transport the largest of the sediment sizes delivered at bankfull discharge by the tributaries".

This new meandering channel therefore transforms the previous deposition reach into a transport reach, but this transformation is only for 1.6 km of the stream system. The sediment moved through this new meandering channel will form new deposits at the end of meandering system resulting in more rapid aggradation than what occurred before the new channel was constructed. Eventually, after several high flows, the aggradation will work its way upstream into the new meandering channel causing its bed to rise and development of avulsions and ultimately a transformation of a deposition reach to a transport reach. This means that the deposition is moved further downstream and will eventually change processes upstream and downstream from the zone of deposition. This project on the East Fork San Juan River will lead to confusion on the part of engineers and environmentalists, especially because of a recent article in *National Geographic* (Special Edition-Water, Nov. 1993) where M. Parfit quotes Rosgen: "Here's a major rule . . . The river must take care of itself".

However, the river was not allowed to take care of itself and was artificially transformed from a natural braided system to a confined meandering system. Wherever river slopes are relatively steep, such as for the East Fork San Juan, the natural pattern is one of braiding as can be derived from Figure 9.36 which indicates thresholds between braiding and meandering. From this plot it is apparent that the natural pattern for a slope of 0.013 would be braided - this pattern can be forced into a meandering pattern if sediment loads are reduced but after a number of years braiding is expected to prevail.

Step 5: Assess Degradation Process

If the geomorphic setting analysis shows that a stream has downcut recently, or if new boulder rapids are exposed, then the stream is probably degrading. Degradation has been defined as the progressive lowering of the general level of a stream channel over a period of years by erosion. The lowering usually extends over long stream reaches, sometimes more than one hundred kilometres, and should not be confused with scour which refers to a local lowering of a stream bed.

Stream bed degradation can pose serious problems for the foundations of bridges or culverts as well as for river training works near the water crossing.

Progressive degradation may be divided into two categories:

X downstream progressing degradation; and
X upstream progressing degradation.

Both types of degradation involve a change of river slope, either imposed or as a result of changes to other hydraulic variables. Generally, upstream progressing degradation proceeds at a much faster rate than downstream progressing degradation because the increased slope results in higher velocities and substantial increases to bed material transport. For downstream progressing degradation, slopes are progressively reduced and bed material transport approaches zero asymptotically, (Galay, 1983 and Figure 9.39).

The variety of river changes and engineering works causing degradation are tabulated in Table 9.2 showing the primary causes and types of changes that commonly occur.

In designing water crossings, the potential for degradation should be carefully assessed in regard to historical ongoing river changes and also to future anticipated river changes, such as cutoffs, or future river works such as weirs. The following procedures are recommended:

Step 5.1: Assessment of Historical Degradation

(a) Examine longitudinal profile, along with airphotos and maps to ascertain changes in water surface over time and changes in channel pattern.
(b) Plot specific gauge curves to assess bed lowering through time (Figure 9.40).
(c) Check each historical type of river change or historical river works shown in Table 9.2 and study the effects of these changes.

Step 5.2: Assessment of Future Degradation

(a) Obtain information from various agencies regarding future plans to change the path of the stream or control levels and then evaluate potential changes to the bed profile.

Step 5.3: Computation of Future Degradation

For downstream progressing degradation, primarily where the cause is stoppage of sediment supply such as by a high dam, the USBR (1982) has developed an empirical relationship whereby the downstream stable bed slope is computed by a three-slope method as shown in Figure 9.41 and by using the following equations for depth of degradation d_g and length of the degraded reach L:

Depth of degradation:

$d_g \quad = (8/13)\ L *)S$

where:

$\Delta S \quad = S_o - S_c$
$S_o \quad$ = Existing bed slope
$S_c \quad$ = Critical, stable bed slope

Length of degradation, L, is frequently the distance to some downstream control point. However, if no control point exists, the length L is assumed and the time to achieve this length is determined by the computation of the total bed-material volume removed over the length L. To compute this volume a bed material rating curve is required along with a flow duration curve.

The stable bed slope after degradation has taken place can be determined from the curve presented in Figure 9.42. For sands a long term stable slope can be obtained from the same Figure 9.42 for a specific bankfull discharge.

For upstream progressing degradation, the process can be similar to a rapid flattening or rotation of the bed slope as shown in Figure 9.43, especially where the cause is lowering of base level, or it may occur as a head-cut process where the height of the knickpoint decreases and the stable slope reached faster than for the rotation process. The head cutting case generally applies to conditions where the bed has some cohesive material or coarse material sizes. The ultimate stable slope can be obtained by using the previously discussed guidelines for downstream progressing degradation with the provision that the head cutting process is more difficult to compute since bed armouring may occur and the degraded length would be shortened if a knickpoint remains as part of the degraded profile.

Table 9.2: Causes of River Bed Degradation

Type of Degradation	Primary Cause	Type of Engineering Works to Cause Degradation
Downstream progressing degradation	• Decrease in discharge of bed material.	• Construction of (high/low) dam. • Excavation of bed material. • Diversion of bed material. • Storage of bed material. • Change in land use.
	• Increase in water discharge, Q.	• Diversion of flow. • Rare floods.
	• Decrease in bed material size, D.	• River process.
	• Others.	• River merging with lake. • Thawing of subsurface permafrost.
Upstream progressing degradation	• Lower base level.	• Drop in lake level. • Drop in level of main river. • Excavation of bed material.
	• Decrease in river length.	• Cutoff. • Channelization and flow regulation. • Horizontal shift of base level. • Stream capture.
	• Removal of control point.	• Natural erosion. • Removal of dam.

An interesting example of upstream progressing degradation has occurred on the Grand River at Brantford, Ontario, where the removal of a navigation weir has swiftly lowered the base level resulting in head cutting. The new channel is now located in the middle of the river as shown in Figure 9.44.

Another example of the degradation and of anticipating changes to fluvial processes over time is shown in Figure 9.45. The figure shows deposition of sand-gravel along the bed of a stream that has various log structures to provide habitat for fish. Prior to the structures, the stream was classified as a stable, straight channel having a clay-silt bed and the dominant fluvial process was transport of sediment through the reach. However, the expansion of an adjacent small street into a wide urban thoroughfare has resulted in the diversion of peak storm runoff directly into the stream. The increase in discharge has resulted in erosion (degradation) along the upper reach as shown in Figure 9.46, and deposition downstream as discussed and shown in the earlier Figure 9.45. The stable, transporting stream reach was transformed to an eroding plus a depositing reach by an inadvertent change in discharge. The deposition along the downstream reach will eventually proceed to a water crossing which will eventually experience a rising of its bed and consequently a drastically reduced flow capacity through the bridge opening.

The previous examples illustrate the importance of considering long-reach fluvial processes such as erosion and deposition; however, in dealing with local improvements for fish habitat, it is necessary to also consider formation and movement of bars. At intakes to restoration channels, sediment should be "encouraged" to move past the intake or, in other words, the location of the intake should be such that sediment does not enter the intake.

Step 6: Assess Aggradation (Deposition) Process

Sediment deposition generally occurs wherever local velocities are being reduced resulting in a subsequent reduction in sediment transport capacity. The assessment of the fluvial processes should indicate natural deposition processes which can be verified by checking specific gauge curves. A visual assessment of a stream reach will readily show the presence of channel bars if deposition is the dominant fluvial process. The various types of bars that generally form in streams are shown in Figure 9.47 and are as follows:

- point bars;
- side bars;
- mid-channel bars; and
- diagonal bars.

An example of a large mid-channel bar is shown in Figure 9.48.

In addition to natural deposition processes, construction-induced changes can bring about deposition as shown in Figure 9.49 and tabulated in Table 9.3. Deposition can progress downstream or upstream depending upon the cause.

A frequent cause of deposition is the raising of base level by the construction of a dam or low weir. In Ontario, a number of streams have low weirs or navigation dams such as shown in Figure 9.50 for the Grand River at Caledonia, Ontario.

An example of aggradation at a proposed bridge crossing is shown in Figure 9.51 where a bridge and a sewer line are being constructed across a relatively small stream. The stream is flowing over a wide alluvial flood plain and has an irregular meander pattern with point bars and mid channel bars of sandy gavel. The design of the bridge calls for protection of the central bridge pier and river banks with riprap thereby resulting in lost habitat for fish. As compensation, large rock clusters and short spurs were planned. However, after more study of the site, it was found that the river bed had aggraded about two metres over the past ten years due to input of sand and gravel from large gravel pits upstream. Therefore, the dominant fluvial process is deposition which will probably continue for decades. In the light of this information, off-channel compensation such as new spawning channels is being considered because any works within the stream would probably be covered over with sediment. There are a variety of good practices that can be used during this type of construction to minimize impacts on fish habitat. For details, refer to the *Environmental Manual: Erosion and Sedimentation Control*, Ontario Ministry of Transportation (1994).

Table 9.3: Causes of River Bed Aggradation

Type of Aggradation	Primary Cause	Type of Engineering Works to Cause Aggradation
Downstream progressing aggradation	• Increase in discharge of bed material.	• Mining operation, waste dumping. • Diversion of bed material. • Change in land use. • Rare flood, wide river section.
	• Decrease in water discharge, Q.	• Diversion of flow.
	• Increase in bed material size, D.	• Rare flood.
	• Reduction in slope, S.	• Alluvial fan formed.
Upstream progressing aggradation	• Rise in base level (bed flatten).	• Dam and weir. • Rise in lake or river. • Build up of bed in tributary at confluence.
	• Increase in discharge of bed material.	• Mining operation, waste dumping.

Step 7: Assess Bank Erosion - Lateral Shifting

Bank erosion results in the lateral shifting of a stream which needs to be assessed whenever water crossings are being considered. Often lateral shifting can be related to stream pattern as graphically illustrated by Chorley et al (1984) where the lateral stability is coupled to the pattern and type of sediment load (Figure 9.52). The braided systems have low stability compared to the meandering or straight channels. An approximate estimate of the annual rate of erosion can be derived from Figure 9.53 which is based on research by Hooke (1980). Basically this chart indicates about one metre of erosion per year for small streams and about 100 m per year for streams having a drainage area of about one million square kilometres.

This tentative estimate is useful in assessing potential lateral shifting, however, bank erosion along meandering channels can take on a variety of forms and their movements have been classified into six main classes (Figure 9.54) by Hooke (1977):

- extension;
- translation;
- rotation;
- enlargement;
- lateral movement; and
- complex change.

The above classes indicate that single channels can shift in a variety of ways which are difficult to predict - one of the more reliable approaches is to study successive airphotos in order to clarify the direction of channel movement near the water crossing.

An example of downstream translation of a gradual meander bend is shown in Figure 9.55. For this case, it is assumed that the bank erosion would commence after a specific threshold discharge is reached and that the amount of erosion would be based on the magnitude of the flood flow.

Another example of bank erosion is shown in Figure 9.56, for the Nith River in Ontario. The banks over a relatively short reach have undergone deposition of bed material in the form of point bars and bank erosion has resulted.

Step 8: Assess Rates of Stream Channel Changes

Step 8.1: Hydrologic Parameters

A detailed presentation of assessing the process of rainfall-runoff and the techniques for arriving at design flows for water crossings is presented in Chapter 8. Here the intention is to highlight the hydrologic information that is pertinent to the understanding of geomorphic processes along a stream reach. The following information should be reviewed.

Flood Hydrograph Obtain various hydrographs from different years, at least one including a flood year in order to assess the range of low to high flows and the duration of the flooding (Environment Canada, 1993). If flow data is not available, then flood hydrographs can be generated as shown in Chapter 8. Most geomorphic changes take place during high flows when sediment loads are high. Sudden peak floods should be noted as shown in Figure 9.57 since most channel changes occur during floods and a large amount of sediment is moved in a short time.

Duration Curves These are important for computing annual as well as individual storm volumes of sediment moved. The rate of degradation is dependent on the time and amount of bed material being removed from a stream reach.

Frequency Curves These are important especially in regions of Canada where snow and rainfall events result in separate data populations as shown in Figure 9.58.

Flood Zoning Maps Some parts of Canada have developed regulatory floods such as Ontario where three zones are presently designated as shown in Figure 9.59. The zones indicate whether the 100-year flood or a specific storm such as the Timmins storm should be used for design of infrastructure as well as for flood plain management. Also, frequencies of the water crossing design flood are discussed further in Chapter 8.

Step 8.2: Sediment Transport

If the stream exhibits instability, then it will probably have a high bed material load which may be necessary to compute, especially if there is a need to know rates of deposition or bed lowering. There exist many references on sediment transport that present a multitude of equations for bed load, total load, bed material load, etc. (ASCE, 1975; Graf, 1971; Simons, Li and Assoc. 1982; Colby, 1964), but for this manual a bed material load equation will be utilized for computing sand loads and a bed load equation will be used for gravel, cobble loads. The Colby chart is plotted for convenient use in Figure 9.60 which shows that the selection of sediment size is critical because the bed material load varies greatly depending upon sediment size.

In some cases, it is also necessary to evaluate when the bed of a stream becomes mobile and this can be done with the use of a chart such as shown in Figure 9.42 or Shields chart shown in Figure 9.61. For gravel-bed rivers, the gravel load can be estimated by using Figure 9.62 and 9.63 which shows a relationship based on the Schoklitsh equation. Also, the suspended sediment loads for Ontario streams appear to be relatively low as shown by a review of sediment production across Canada by Stichling (1973) and by the chart in Figure 9.64 reproduced from another chapter of this manual.

Step 8.3: Resistance to Flow

Roughness is estimated through various techniques and is often referred to as resistance to flow - however, it must be stressed that the roughness is not a single value but varies with the mobility of the river bed as well as the depth of flow and other more minor factors.

The roughness in a river channel can vary widely and its value depends on a large number of factors (Chow, 1959):

- surface roughness;
- vegetation;
- channel irregularity;
- channel alignment;
- obstructions;
- size and shape of channels;
- stage and discharge;
- silting and scouring;
- seasonal change; and
- suspended material and bed load.

There are several different ways of expressing roughness depending upon the flow equation being used. One method is Manning equation,

$$V = (1/n) * R^{2/3} * S^{1/2}$$

where:

V	= Velocity of flow, m/s
n	= Manning's roughness
R	= Hydraulic radius, m
S	= Slope, m/m.

The values for the Manning's roughness coefficient are presented in Design Chart 2.01.

Step 9: Reiterate Degradation and Aggradation Assessment

After assessing historical vertical and/or lateral changes to the stream channel, another check should be made regarding the influence of degradation or aggradation on future lateral shifting. If a channel is found to be degrading, then the height of exposed river bank increases which leads to eventual caving of the bank at a faster rate than one would predict. An interesting example of this occurrence was found on the Solo River, Indonesia where eroding bank heights were all above 10 m in height and bank erosion occurred soon after long-reach degradation had occurred (Monenco, 1986). Another common reason for accelerated bank erosion is deposition of bed-material on large mid-channel bars which forces high velocity currents into the river banks. It seems that vertical changes in the level of the river bed induce bank erosion. Therefore, the assessment of bank erosion (Step 7) may need to be revised in the light of anticipated changes to the river bed.

After establishing the stability of the stream channel in its existing state, consideration of future trends is essential so that future channel changes can be anticipated. Information from other agencies dealing with the stream will allow the designer to predict trends in changes to the stream bed and banks; for example the lowering of a lake level will result in future degradation and possible failure of bridge or culvert foundations. The previous text deals with establishing existing and historical behaviour of the stream channel - the future response also requires consideration.

Step 10: Finalize Conclusions for Planners and Designers

Generally, a summary of the stream stability status along with predictions for the future stability are the result of the stream stability analysis. Based on the analysis, the geomorphologist/engineer should rank the proposed crossing options.

Example No. 1

Grand River near Brantford, Ontario.

A preliminary study involves the construction of a road from the Intersection of Hwy 53 and Hwy 2 to the Brantford Airport on the south side of the Grand River. An early stage of the study involves a geomorphic assessment of the proposed crossing.

Step 1: Assess geomorphic setting

Available Information:

- Topographic contour map, 1:50,000 scale (Figure 9.65)
- Aerial photos, 1972 and 1978 (Figure 9.66)
- Oblique photos from overflight (Figure 9.67)
- Ground photos from field trip (Figure 9.68)
- Geographic features data sheets (Figure 9.69 a, b, c)

The brief investigation indicates that the site is located in a partially confined valley and the river is entrenched into the flood plain as indicated by boulder rapids and bedrock.

Step 2: Evaluate environmental conditions

The water crossing site should be located at the upstream end of the existing island as this site shows a boulder/bedrock river bed. The trees along the south river bank provide shade for fish and the island provides nutrients whenever higher flows overtop the island.

The fish habitat may be improved by cabling root wads and trees to a pier which could be located on the island.

Step 3: Evaluate fluvial processes

From an assessment of the channel shown in airphotos and from analysis of the crater-surface slope profile, it appears that the bed is controlled by boulders and/or bedrock and the slope is somewhat steep (about 0.003). There are no bars in the reach and since the channel appears to be entrenched, it is therefore not aggrading. Also, because of boulders, the bed is not degrading rapidly.

Step 4: Assess channel characteristics

From preliminary appraisal, the river channel appears stable. Using an estimated discharge of 200 m^3/s and the regime charts (Figures 9.34a, 9.34b) one obtains:

Regime bankfull width = 70 m
Actual bankfull width = 110 m

Regime bankfull depth = 2.7 m
Actual bankfull depth = 3 m

The water-surface slope profile indicates that there is boulder and/or bedrock control in the vicinity of the crossing and the local slope is relatively steep. The slope profile downstream is much flatter and is controlled by a clay, glacial deposit.

The channel has a stable channel relatively close to regime dimensions. The meander pattern is irregular and is partly controlled by the confining valley.

Since the channel is relatively stable and the bed shows boulder rapids, Steps 5 and 6 can be bypassed.

Step 7: Assess lateral shifting processes

From a comparison of airphotos dated 1972 and 1978, see Figure 9.65. It becomes evident that lateral shifting is virtually non-existent. The vegetation along the banks has not changed.

Steps 8 to 9: Rates of geomorphic changes

Steps not necessary since there is little vertical or lateral movement of the channel.

Step 10: Conclusions

The location of the crossing should be at the upstream end of the island where boulders/bedrock are in evidence. A pier could be located on the island and could be used to improve fish habitat by cabling of root wads, trees, etc. to the pier. The channel at this location is stable and abutments should be placed back from the channel as has been done for other recent bridges along the Grand River (Figure 9.70).

Example No. 2

Northern River in Canada

A proposed northern highway is to cross a tortuous, meandering river. An early stage of the study involves an assessment of stream geomorphology.

Step 1: Assess geomorphic setting

Available Information:

- Topography contour map (Figure. 9.71)
- Aerial photos, 1949 and 1970 (Figure 9.72)
- Oblique photos from overflight, (Figures 9.73, 9.74, 9.75, 9.76)
- Bed-material is sand and gravel with D_{50} = 5 mm, D_{40} = 4 mm and D_{90} = 30 mm.
- Slope profile (Figure 9.77)
- Geographic features data sheets (Figure 9.78a, b).

A brief assessment of the data sheets indicates that the river flows over a broad, shallow alluvial plain and that there is relatively little constraint to lateral movement.

Step 2: Evaluate environmental conditions

The river is in a remote region with little interference from people. The channel is shaded by trees and a fish biologist should assess species and numbers of fish.

Step 3: Evaluate fluvial processes

From an examination of the airphotos and the slope profile, it appears that the river reach is undergoing slow deposition. The extensive point bars and side-channel bars indicate a significant supply of sand and gravel, but bank erosion is slow from 1949 to 1970. There exists a waterfall (Figure 9.74) about 2,500 m downstream from the proposed crossing which could result in upstream degradation (head cutting) if a meander loop cutoff occurs upstream from the waterfall.

Step 4: Assess channel characteristics

For a gravel river, the "regime" width is slightly less than shown on the regime charts of Figure 9.34. Using a bankfulll discharge of Q = 142 m^3/s:

Regime bankfull width = 52 m
Actual bankfull width = 37 m
The depths are not known.

The stable slope for a "regime" gravel river can be approximated from Figure 9.42 which yields:
Slope = 0.0004
Actual slope = 0.0007

The actual slope is steeper than a stable slope which partially indicates that the channel transports some bed-material. The bedrock outcrop at the waterfall is a local control on slope.

Step 5: Assess degradation process

There is a potential for a loop-cut which could result in a lowering of base level and possible head cutting at the water crossing site.

In order to assess the possible length of future degradation, assume that eventually the bed at station 4,000 m will be lowered by 3 m and that the stable slope could be achieved when gravel particles smaller than D_{90} are removed. Assuming the bed becomes paved and $D_{40} = D_{90}$, using the chart on Figure 9.42 results in a stable slope = about 0.0004. Plotting this slope on the slope profile shows degradation proceeding up as far as station 2,100 m, somewhat below the proposed water crossing.

Step 6: Assess aggradation process

There may be some future aggradation downstream from the potential loop-cut once degradation starts and this may result in a slow-down to the upstream progressing degradation.

Step 7: Assess lateral shifting process

Figure 9.71 shows only minor changes to channel location or bank erosion over 21 years.

Step 8: Rates of geomorphic change

The upstream progressing degradation would depend on the rate of gravel transport which will initially be high as head cutting starts and reduces with time. A flow duration curve and a gravel transport rating curve is necessary to approximate the time for arrival at a stable slope.

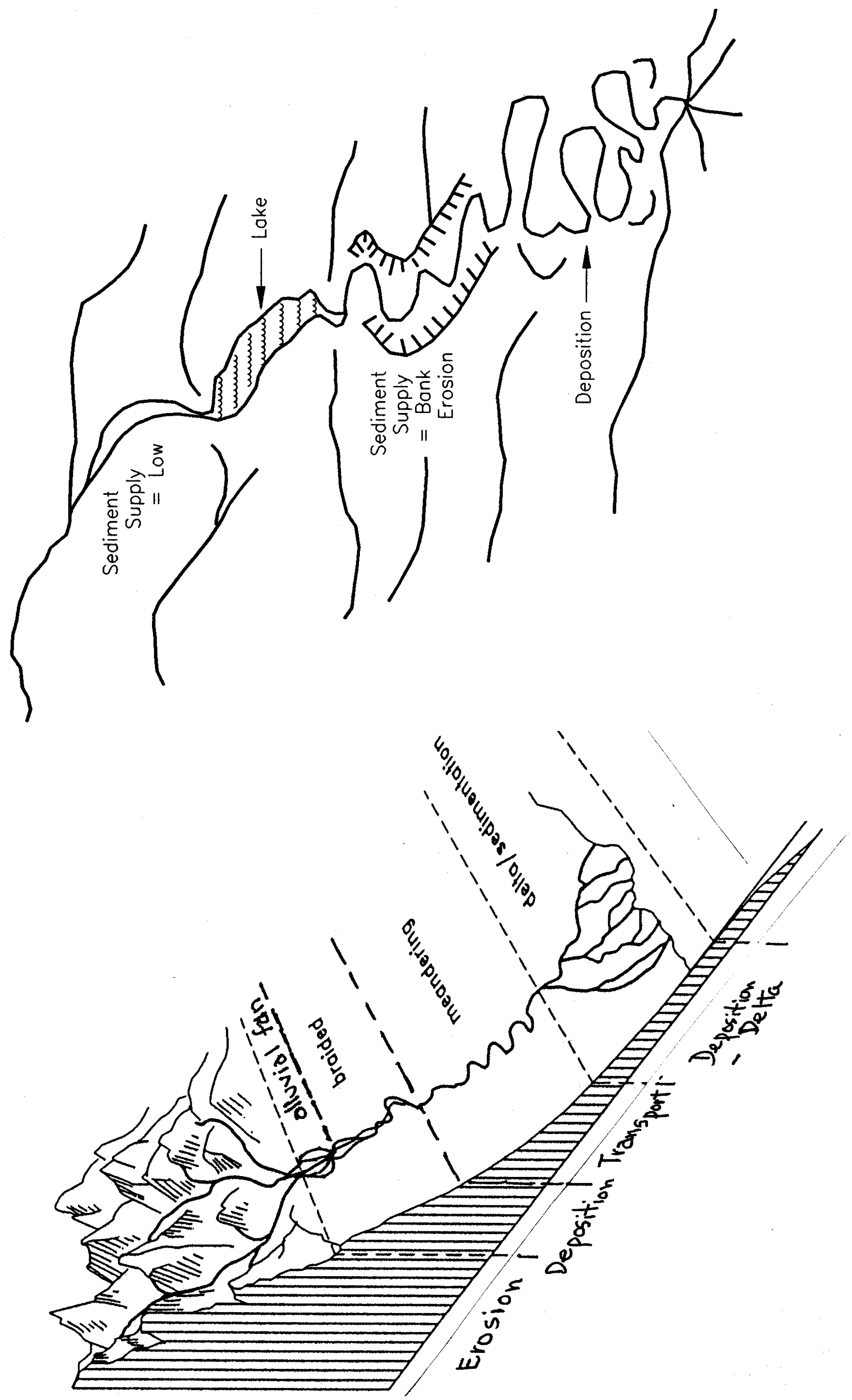
Step 9: Re-iterate vertical and lateral processes

This step could involve assessment of downstream deposition as well as the path of head cutting to anticipate the potential new channel path.

Step 10: Conclusions

The location of the crossing is at a river reach that is laterally stable and potential degradation near a downstream water fall should not reach the crossing site.

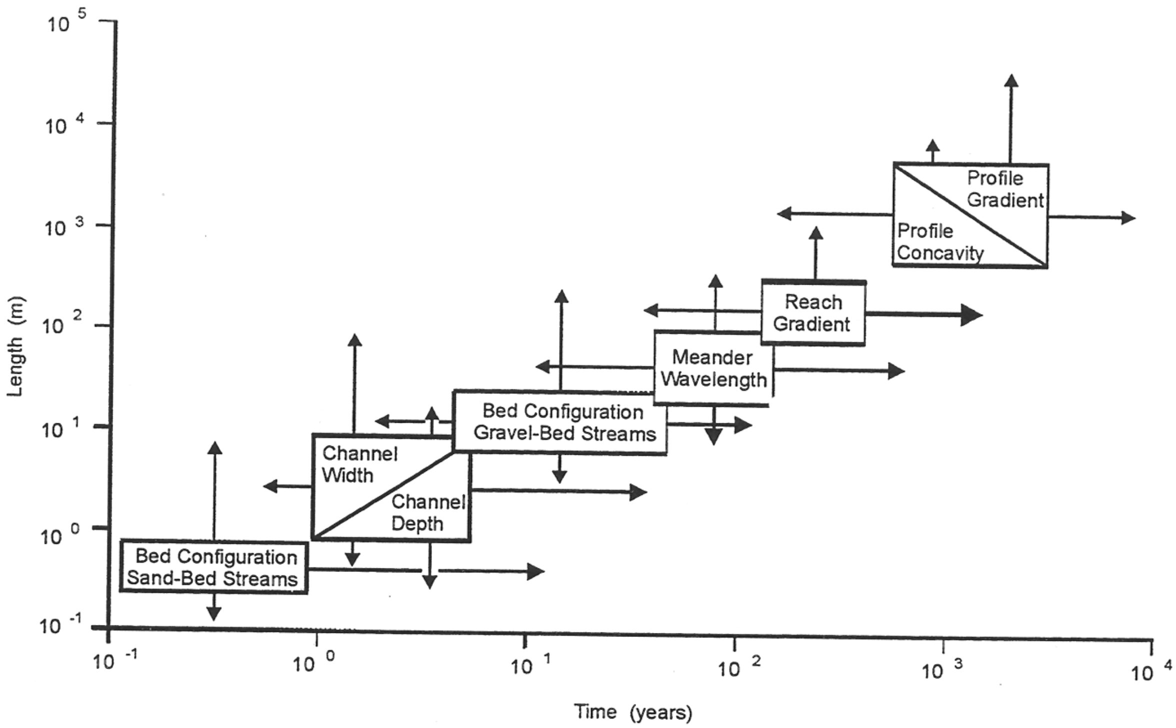
Figure 9.01: Fluvial Processes Along Stream Systems



(b) Channel Types in Ontario

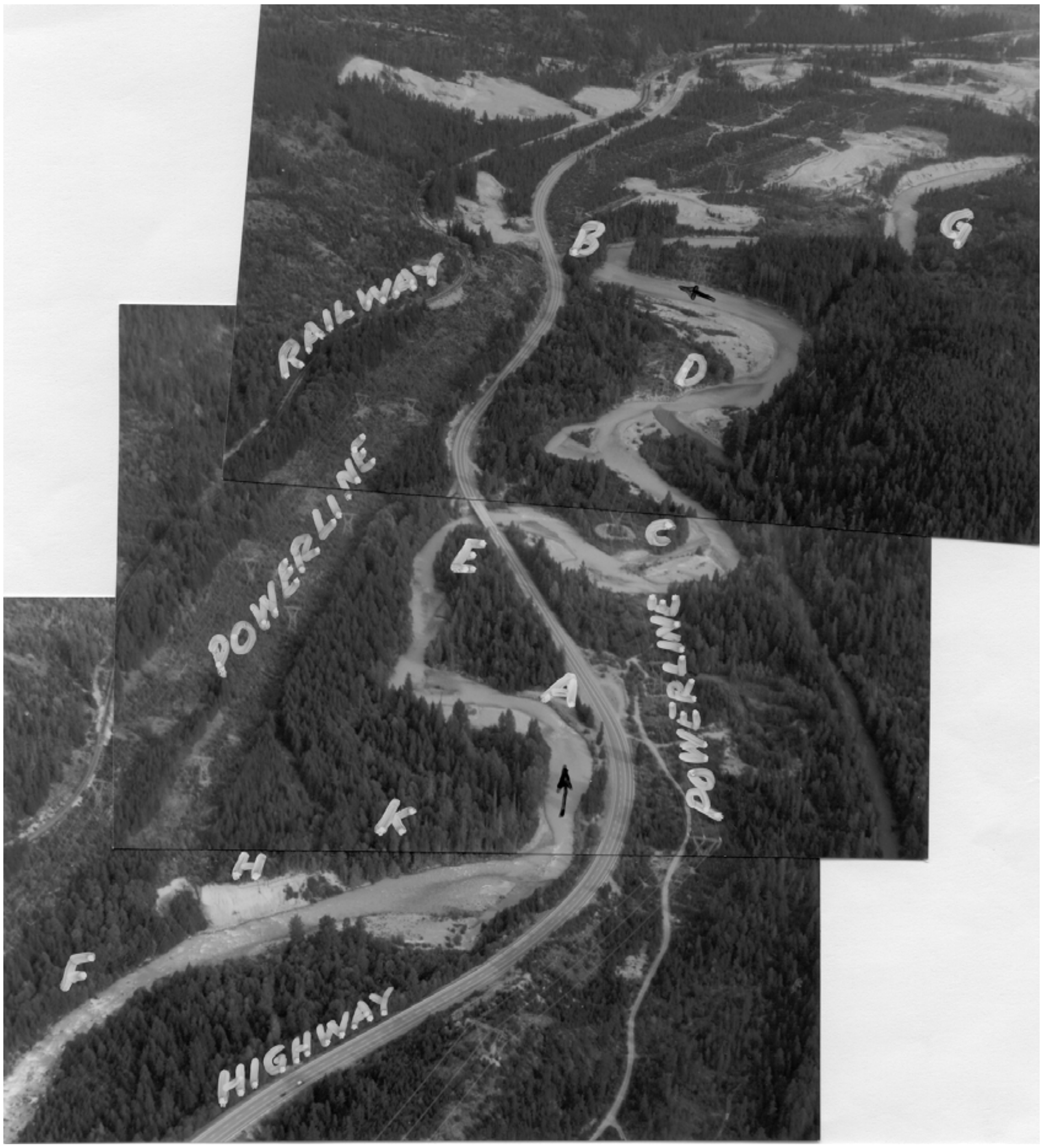
(a) Drainage Basin Zones and Some Channel Types

Figure 9.02: Time Scales for Adjustment of Stream Channels.



Source: (After Knighton, 1984)

Figure 9.03: Downstream View of a Transport Corridor along a Stream Valley Involving Highway, Railway, Powerline, Gravel Pits Interactive with a Meandering Stream.



Eroding banks are shown at A and B and riprap protection of powerline foundations are shown at C and D. Bank erosion at H is supplying sediment to the unstable reach from H to G. A large gravel pit is shown opposite to G and near B.

Figure 9.04: Foundation of Bridge Piers Removed by Upstream Progressing Degradation Caused by Gravel Excavation Downstream From the Bridge.

Figure 9.05: Aggradation of Stream Channel Through Delta Reach.

(a) View of delta from air.

(b) Aggradation at railway crossing. Note the spoil pile on the left bank after excavations of the river bed.

The stream is crossed by a road bridge followed by a railway bridge.

Figure 9.06: Eroding Bend Protected by Riprap.

(a) Natural eroding bend with pools, nutrients along bank and logs in channel.

(b) Protected bank with no pools.

Figure 9.07: Pool and Riffle Sequence Developed along Straight Channel with a Gravel-Sand Bed Material

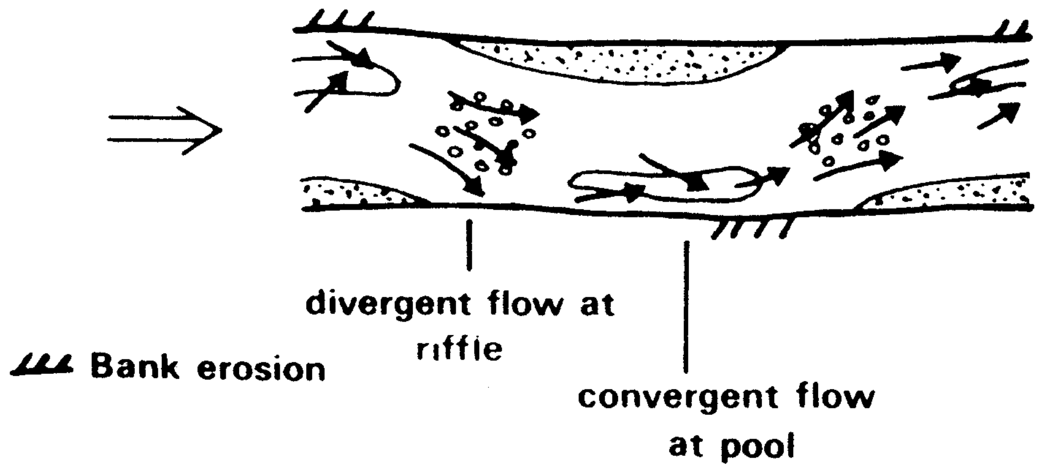


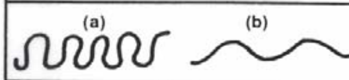
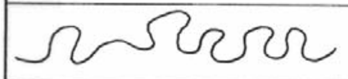
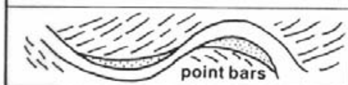
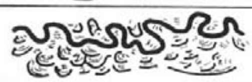
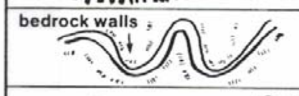
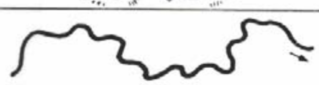
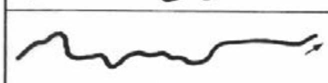
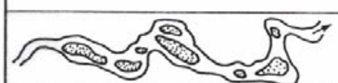
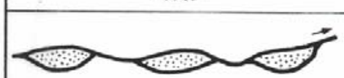
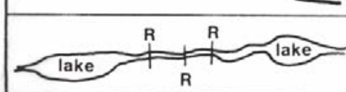
Figure 9.08: Procedure for Assessing Stream Stability.

1. **ASSESSING GEOMORPHIC SETTING**
2. **ASSESS STREAM ENVIRONMENT (FISH HABITAT)**
3. **EVALUATE WATER-SEDIMENT PROCESSES**

4. STABLE CHANNEL **5. DEGRADION** **6. AGGRADATION** **7. BANK EROSION (LATERAL SHIFTING)**

8. **ASSESS RATES OF GEOMORPHIC CHANGES**
   - **a) Hydrologic Parameters**
   - **b) Sediment Transport**
   - **c) Resistance to Flow**
9. **RE-ITERATE VERTICAL AND LATERAL PROCESSES**
10. **CONCLUSIONS FOR PLANNERS**

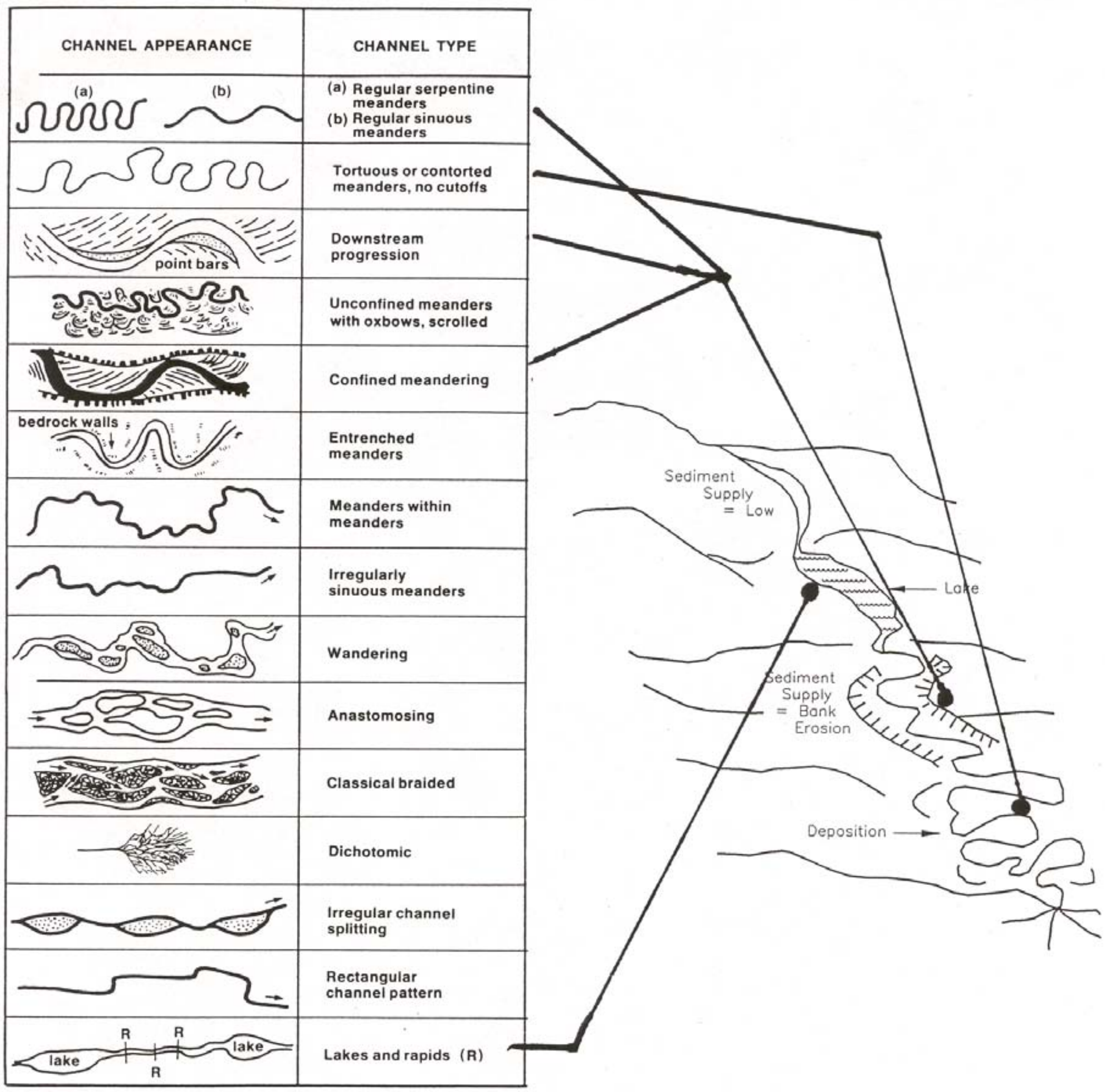
Figure 9.09: Channel Pattern Variations

CHANNEL APPEARANCE	CHANNEL TYPE	TYPICAL ENVIRONMENT	TYPICAL BED AND BANK MATERIALS
	(a) Regular serpentine meanders (b) Regular sinuous meanders	Lacustrine plain	Uniform cohesive materials
	Tortuous or contorted meanders, no cutoffs	Misfit stream in glacial spillway channel	Uniform cohesive materials
	Downstream progression	Sand-filled meltwater channel	Slightly cohesive top stratum over sands
	Unconfined meanders with oxbows, scrolled	Sandy to silty deltas and alluvial floodplains	Slightly cohesive top stratum over sands
	Confined meandering	Cohesive top strata over sand substratum in steep-walled trench	Slightly cohesive top stratum over sands
	Entrenched meanders	Hard till or uniform rock	Till, boulders, soft rock
	Meanders within meanders	Underfit streams in large glacial stream spillways	Cohesive materials
	Irregularly sinuous meanders	Thin till over bedrock in plains	Hard and softer materials
	Wandering	Foothills and mountain valleys	Cobble-veneered sand
	Anastomosing	Foothills, plains. Sand bed or gravel paved rivers	Sand and gravel
	Classical braided	Glacial outwash. Foothills	Sand and gravel
	Dichotomic	Alluvial cones and fans	Gravel, sand, silt
	Irregular channel splitting	Large rivers in bedrock	Alternate sand, gravel and rock
	Rectangular channel pattern	Jointed rocks, mostly flat-lying sedimentary rocks	Rock
	Lakes and rapids (R)	Till-veneered Shield terrain	Till, cobbles, boulders, hard rock

Some of the many channel pattern variations.

Source: (after Mollard Janes, 1984).

Figure 9.10: Channel Patterns Common to Ontario.



Some of the many channel pattern variations.

Figure 9.11: Narrow Bedrock Reach with Several Bridge Crossings.

Figure 9.12: Boulder-lined Reach with Riprap Protection at Bridge

Flow is right to left.

Figure 9.13: Boulder-lined Stream along the Same Reach as Shown in Figure 9.12

Figure 9.14: Physiographic Regions of Canada.

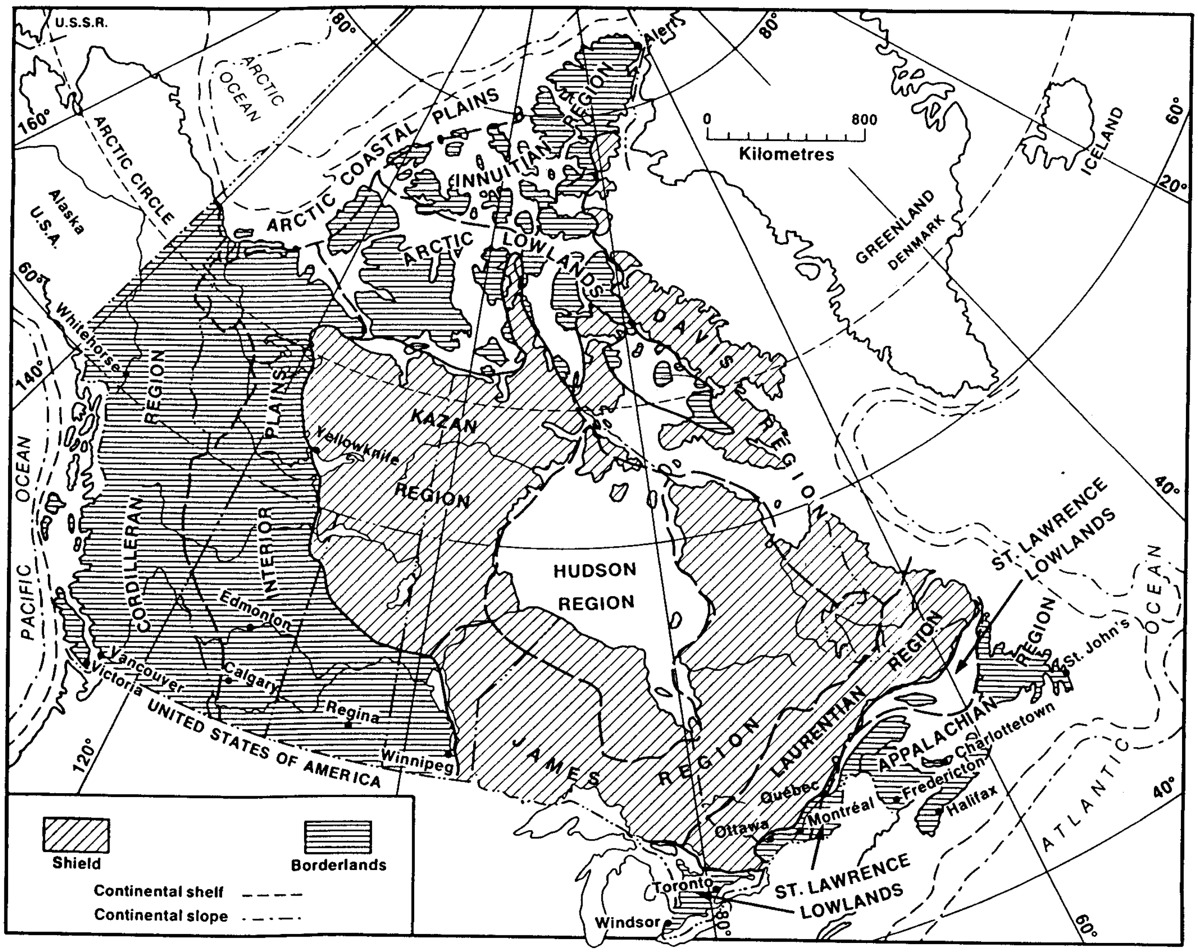
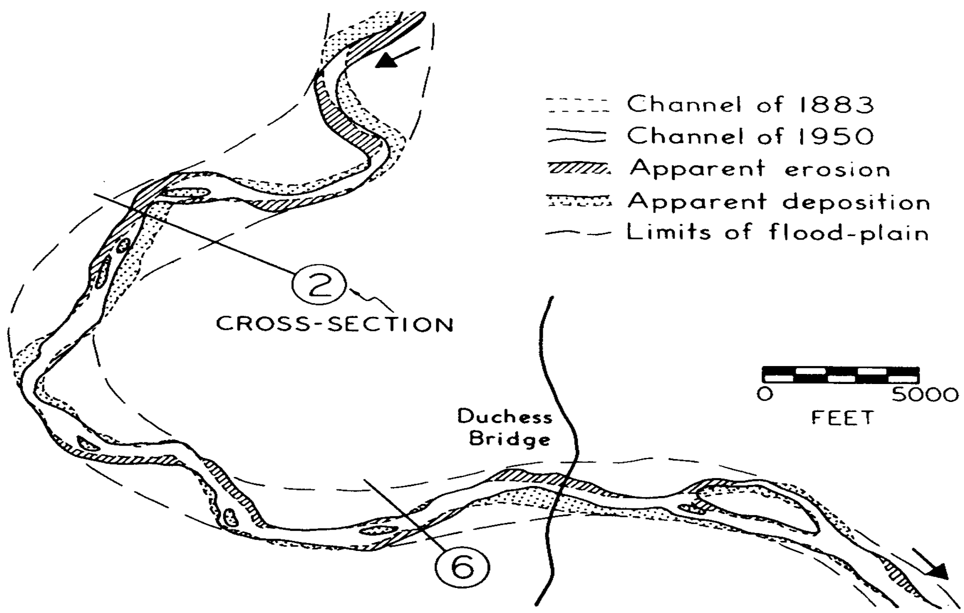


Figure 9.15: Albany River in Northern Ontario.

Figure 9.16: Burntwood River in Manitoba Flowing over Bedrock Rapids.

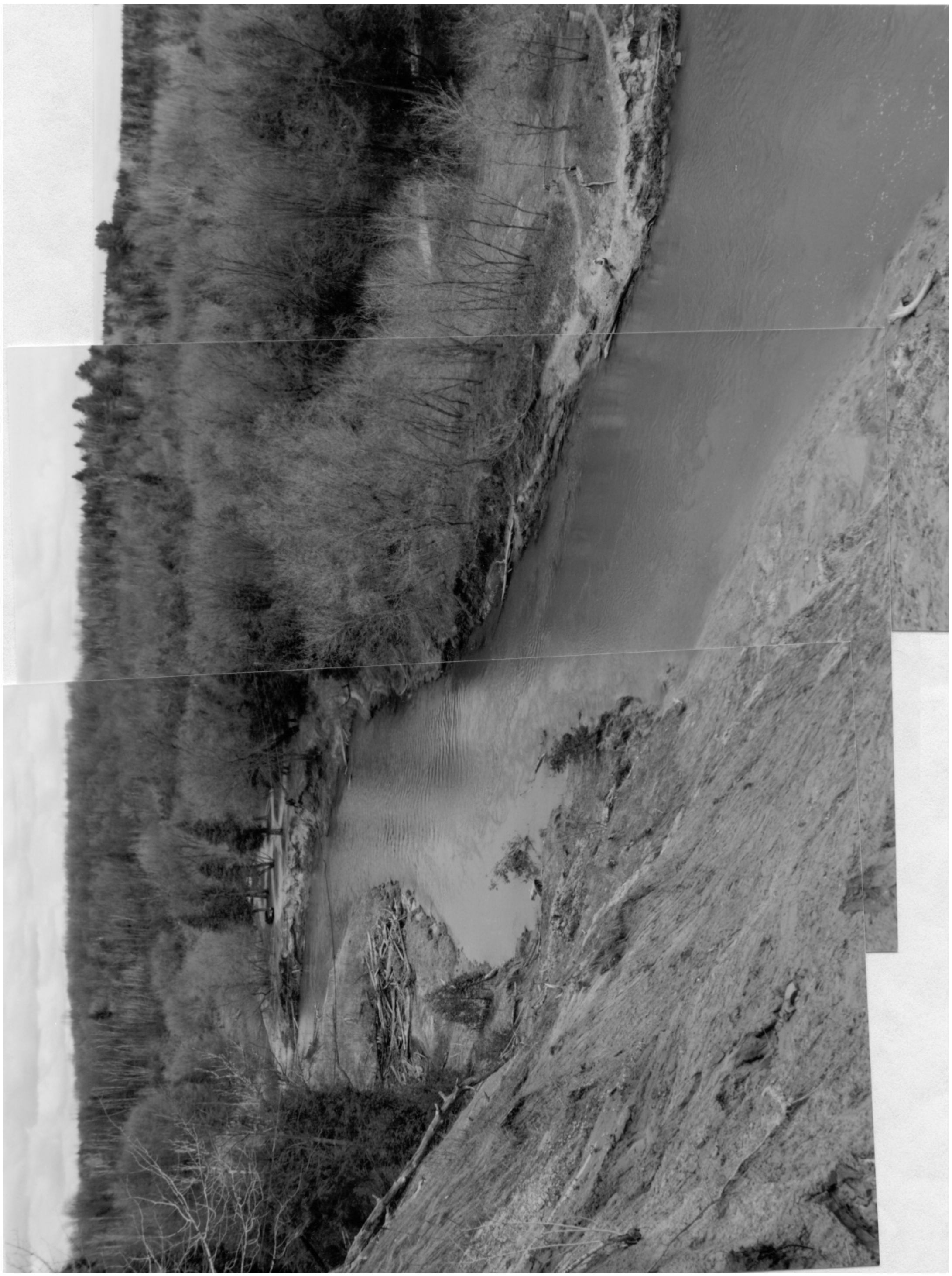
Figure 9.17: Confined Meander Pattern within an Intermediate Width Valley.

Figure 9.18: Translation of Meander Pattern in a Downstream Direction.



Source: (After Neill and Galay, 1967)

Figure 9.19: Nottawasaga River, Ontario Flowing in an Intermediate Width Valley.



The left bank of the stream is impinging onto a unstable valley wall which is slumping and feeding sediment into the stream. View is downstream.

Figure 9.20: Stream in Wide Valley Having Alluvial Fans, Terraces and a Wandering Unstable Main Flow Channel.

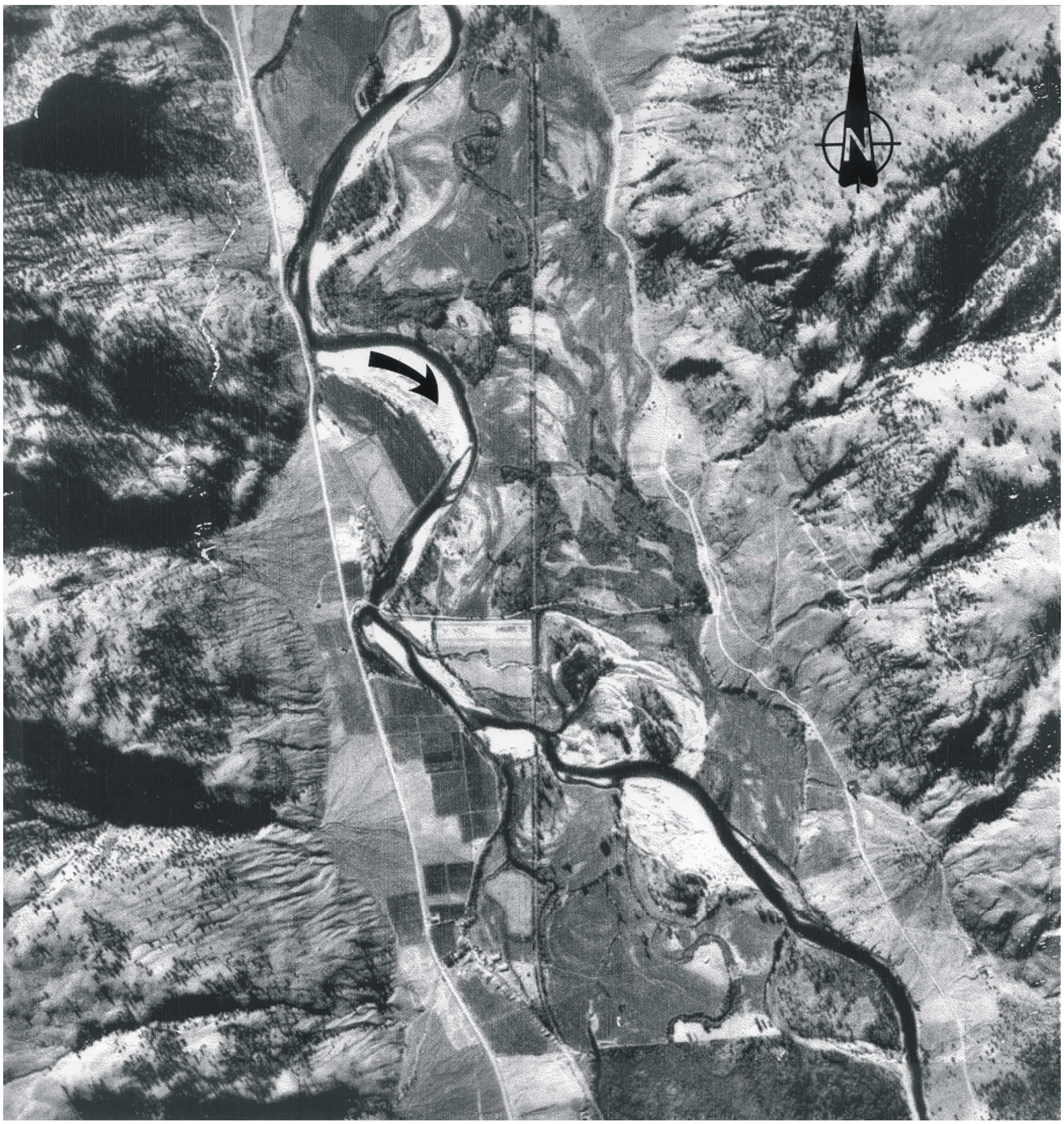
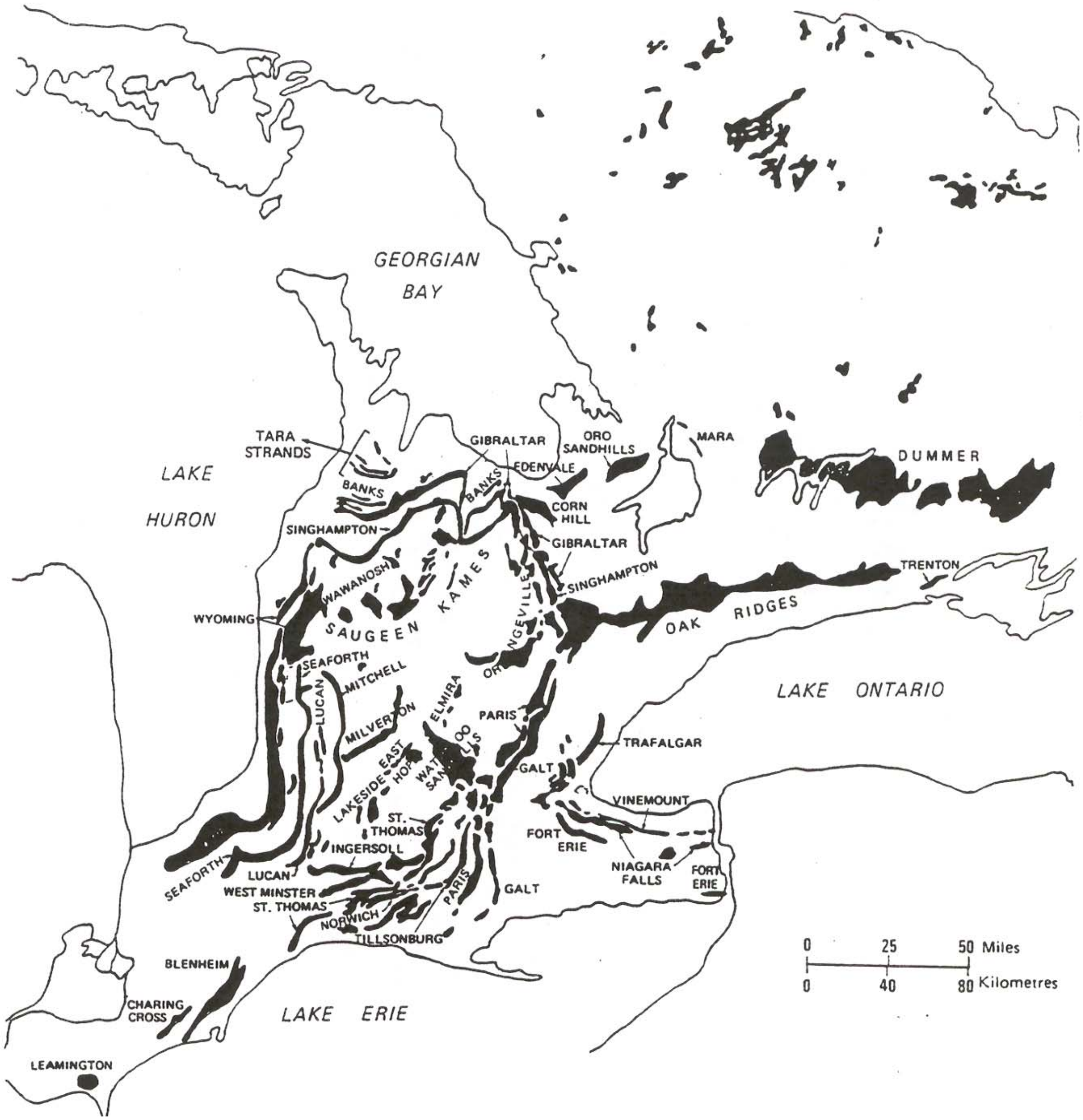


Figure 9.21: Partially Confined Alluvial Fan Emerging from Narrow Gorge.

Figure 9.22: The Grand River in Southern Ontario.

This meandering river is flowing over glacial material and through moraines.

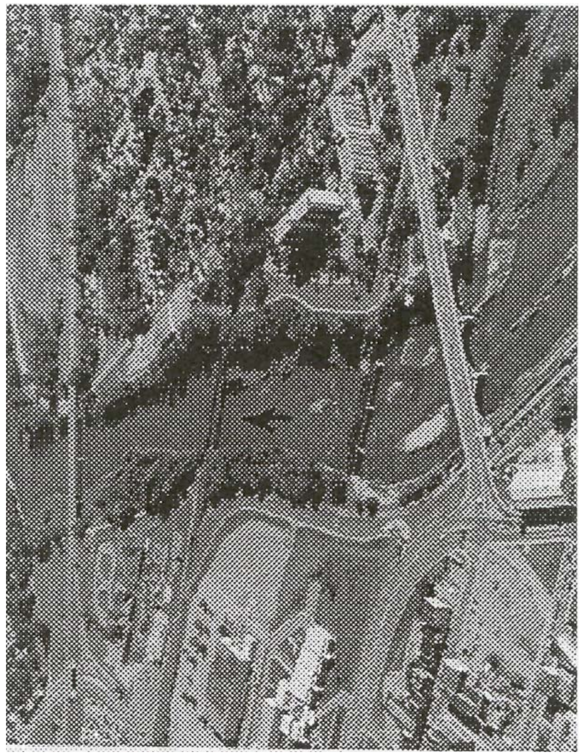
Figure 9.23: Moraines of Southern Ontario.



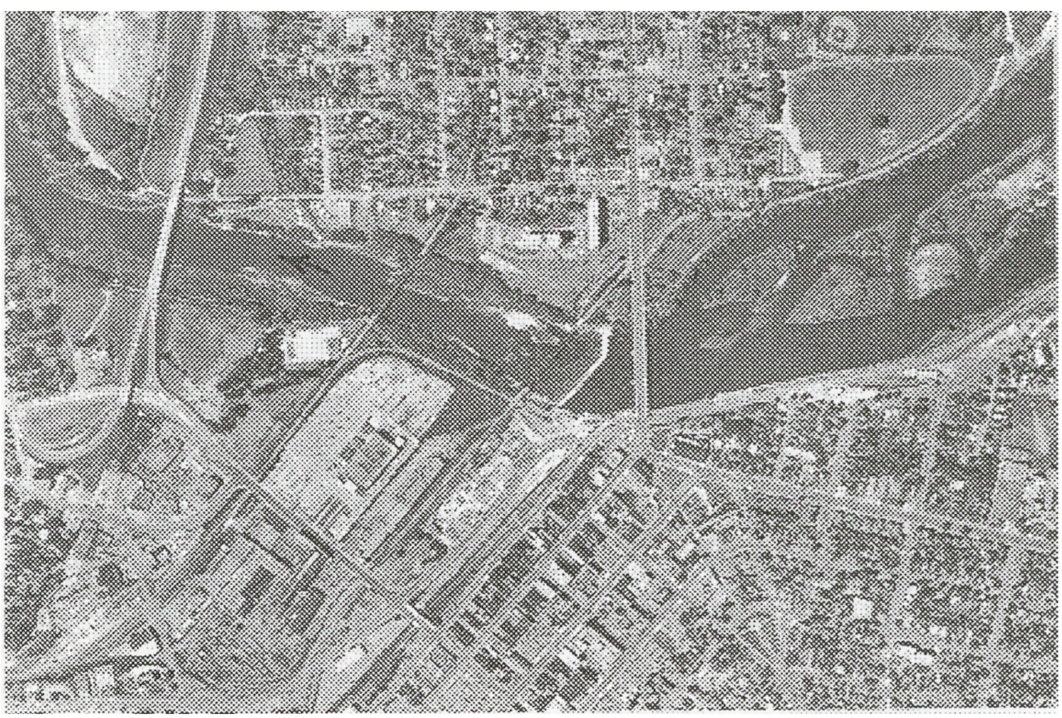
Moraines of Southern Ontario

Source: (After Chapman and Putnam, 1984)

Figure 9.24: Grand River at Brantford, Ontario.



(b) Photo - 1994



(a) Air photo - 1987

Flow is from bottom to top. The weir is shown in 1987 has been removed and the channel upstream from the bridge has headcut in an upstream direction.

Figure 9.25: Goulais River, Ontario.

Flow is right to left.
The river flow over silt/clay deposits and is slowly abandoning meander loops.

Figure 9.26: Goulais River Delta.

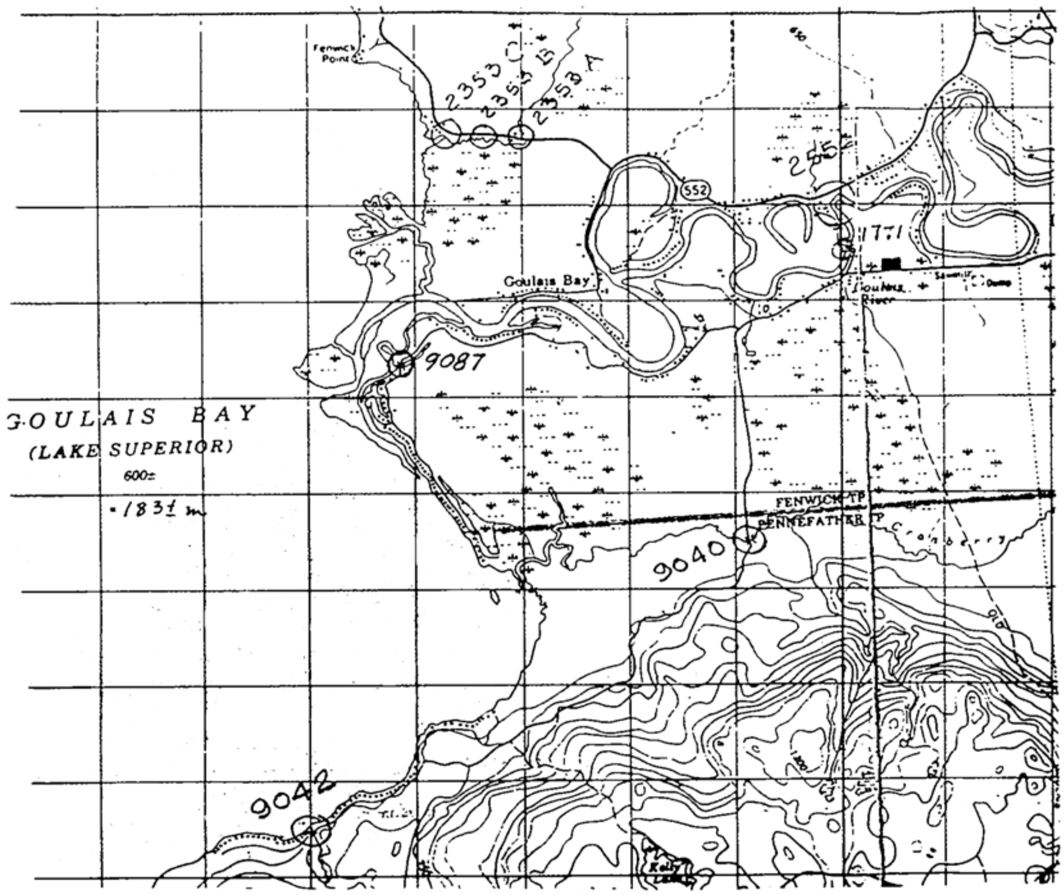


Figure 9.27: Riparian and Fisheries Sensitive Zones.

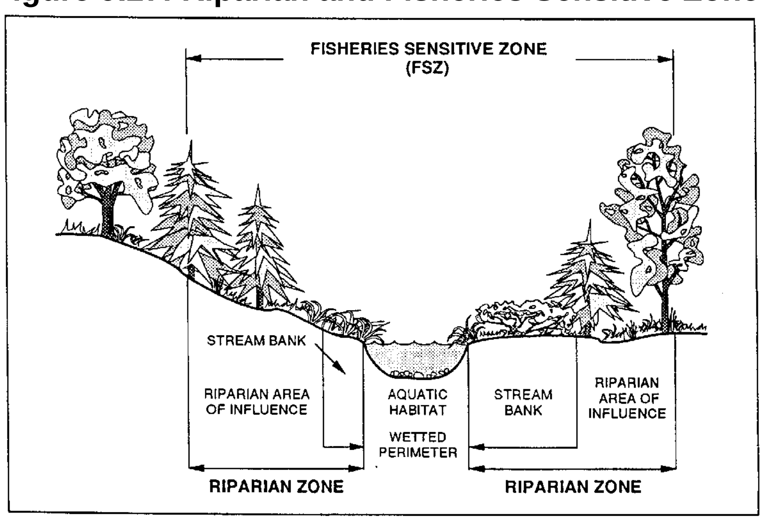
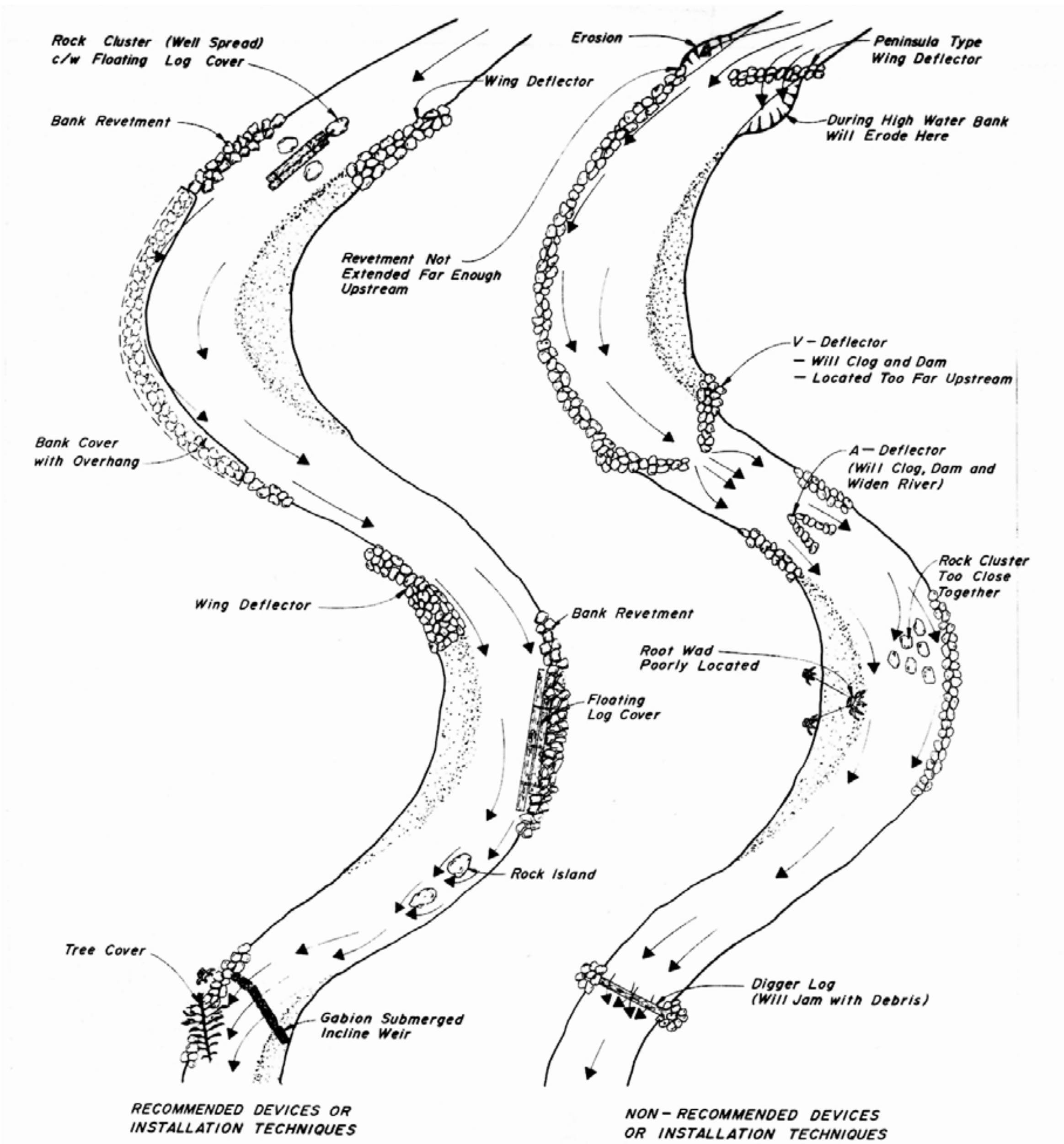


Figure 9.28: Pool and Riffle Sequence along Unstable, Grand River.

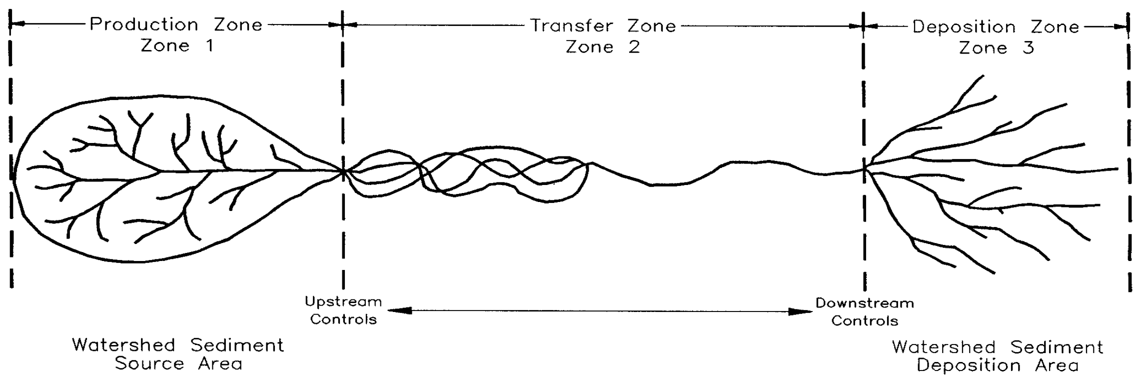
Figure 9.29: Various Recommended and Non-recommended Stream Enhancement Techniques.



Source: (Adapted from White.)

Note: All of the features depicted as recommended devices may not be suitable for all stream types. Refer to Environmental Manual (draft): Fisheries for more details.

Figure 9.30: Idealized Fluvial System.



Source: (After Schumm, 1977)

Figure 9.31: Idealized Sediment-Water System.

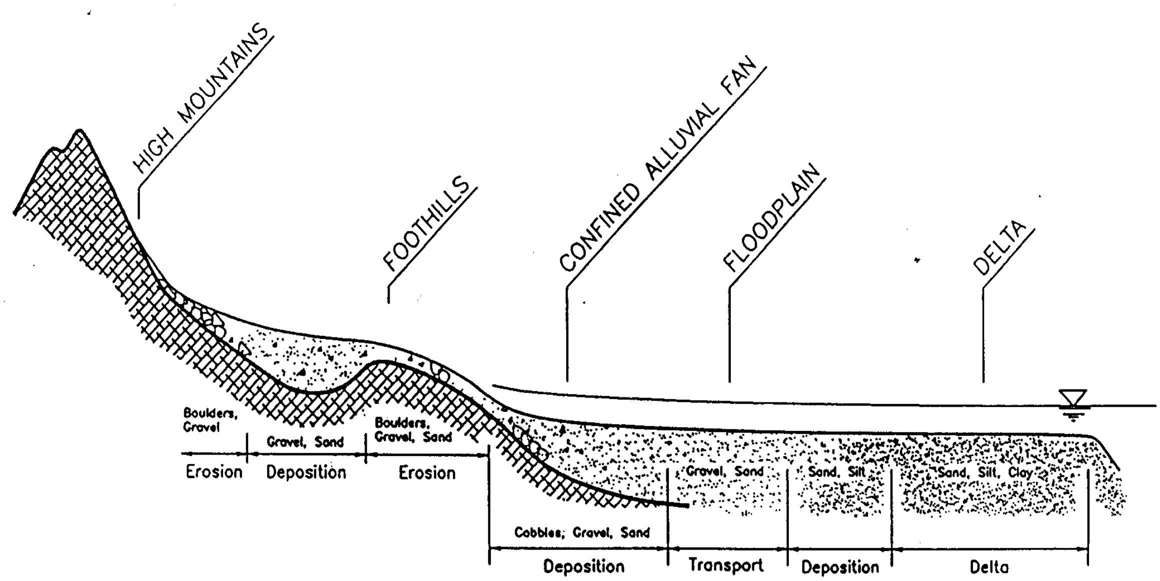


Figure 9.32: Longitudinal Profile of Stream (Skykomish River).

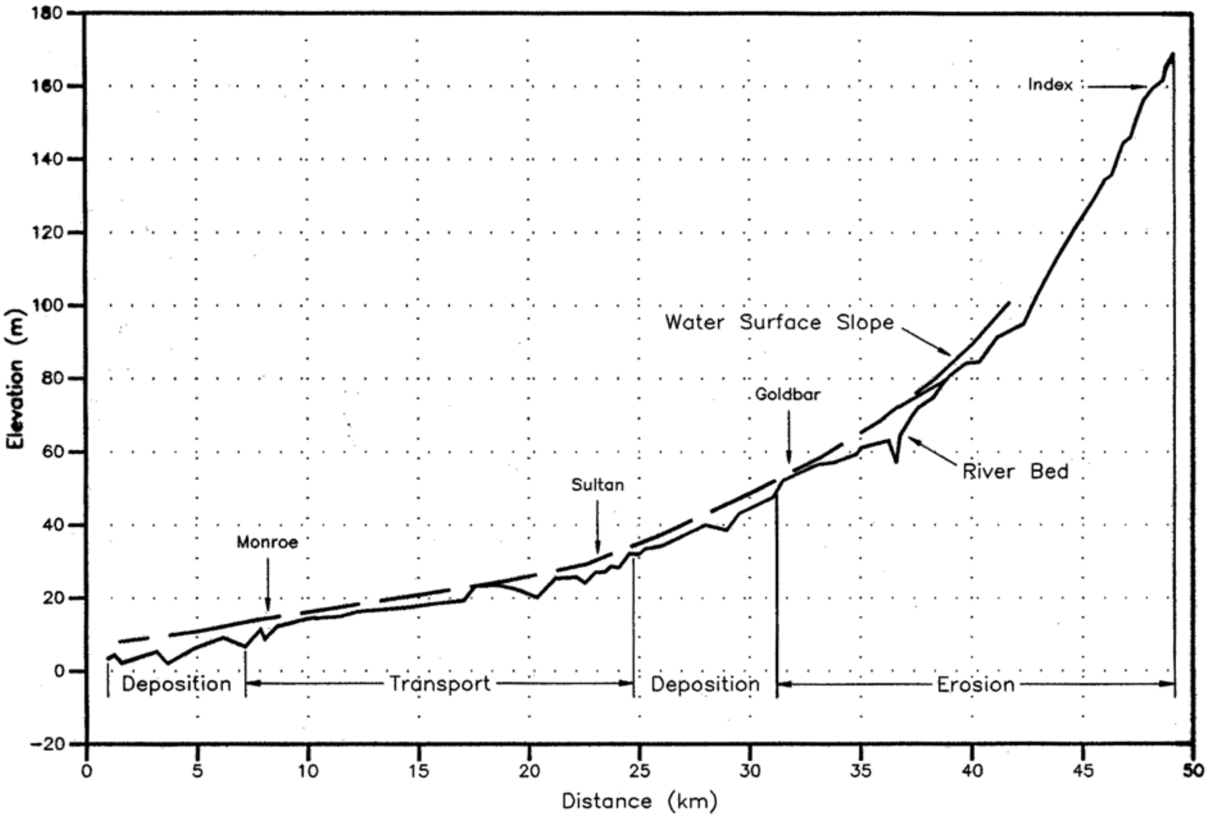
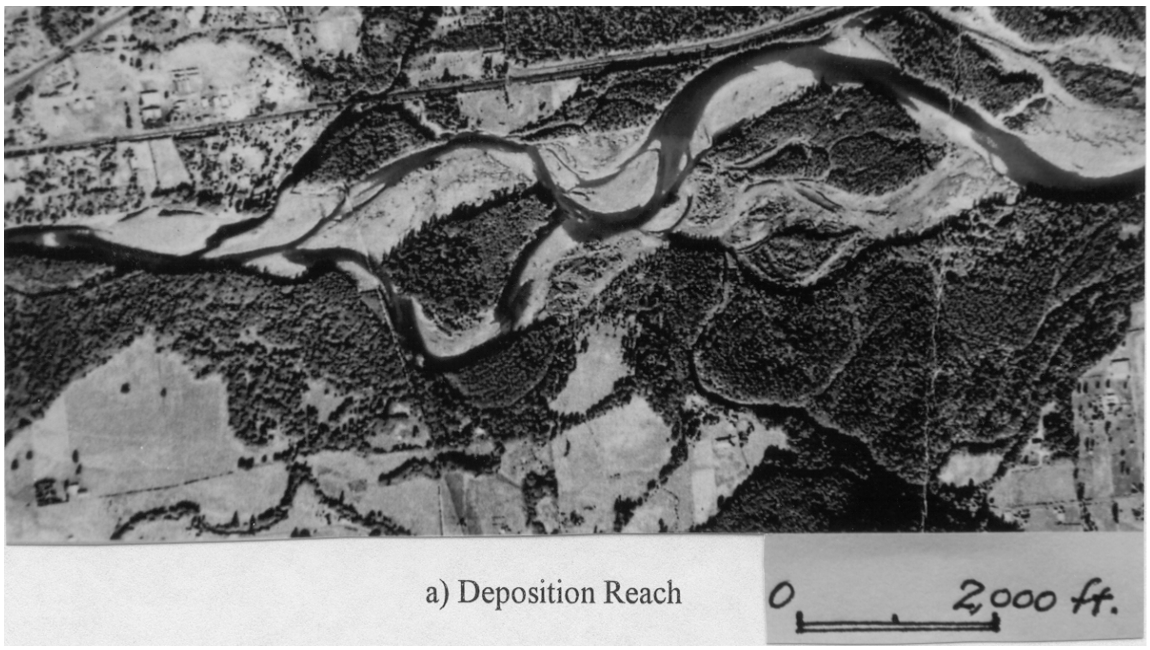


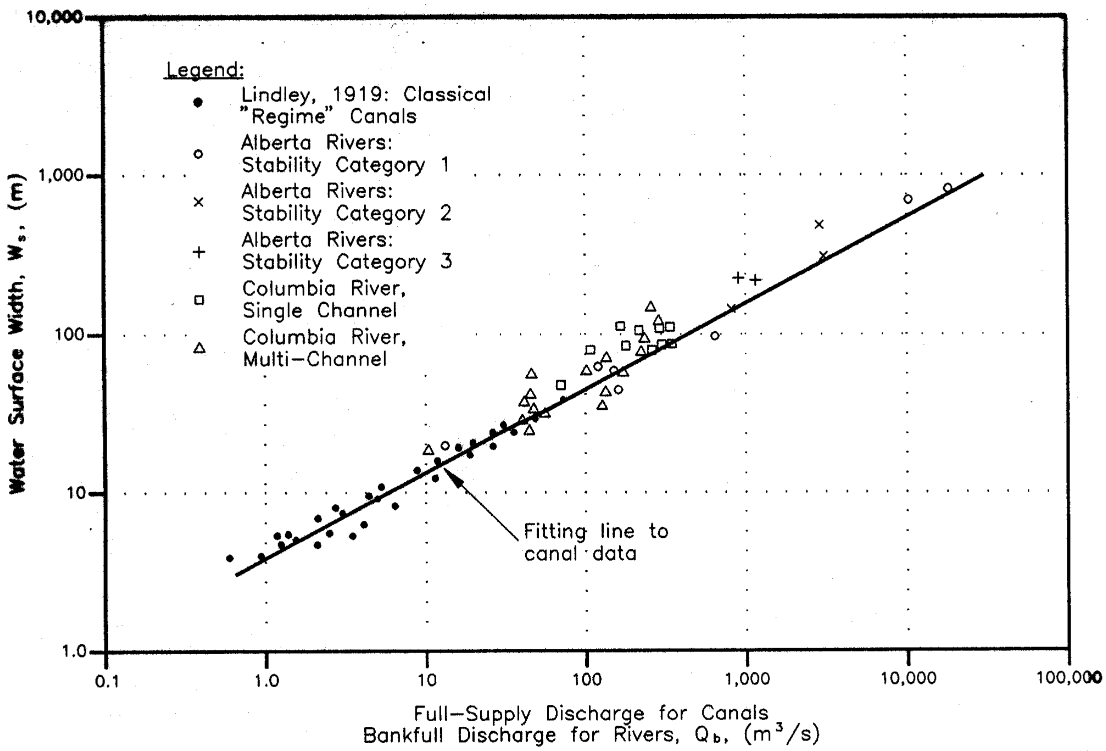
Figure 9.33: Deposition Zone and Stable, Transport Reach for Longitudinal Profile Shown in Figure 9.32.



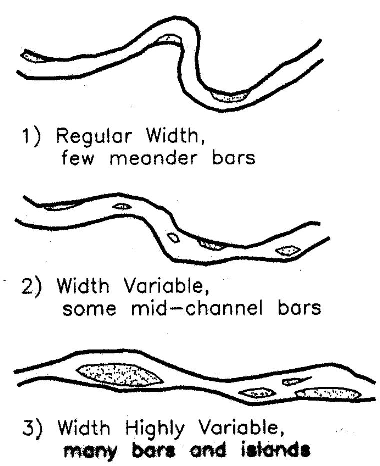
a) Deposition Reach

b) Transport Reach

Figure 9.34a: Stable Channel Characteristics.



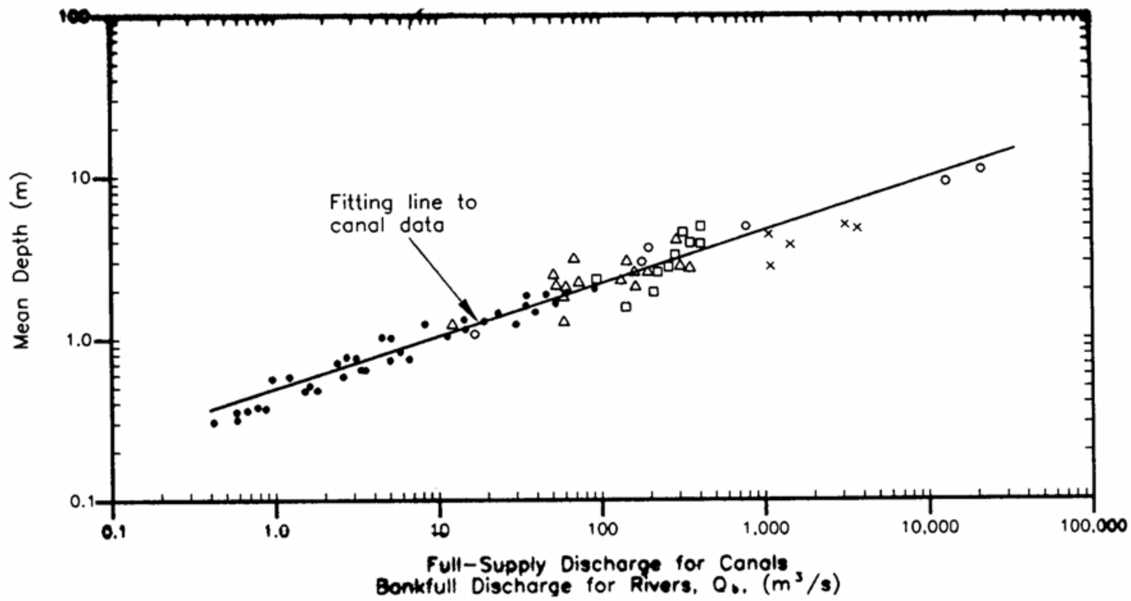
a) Water Surface Width vs. Channel-Forming Discharge for Sand-Bed Canals and Rivers



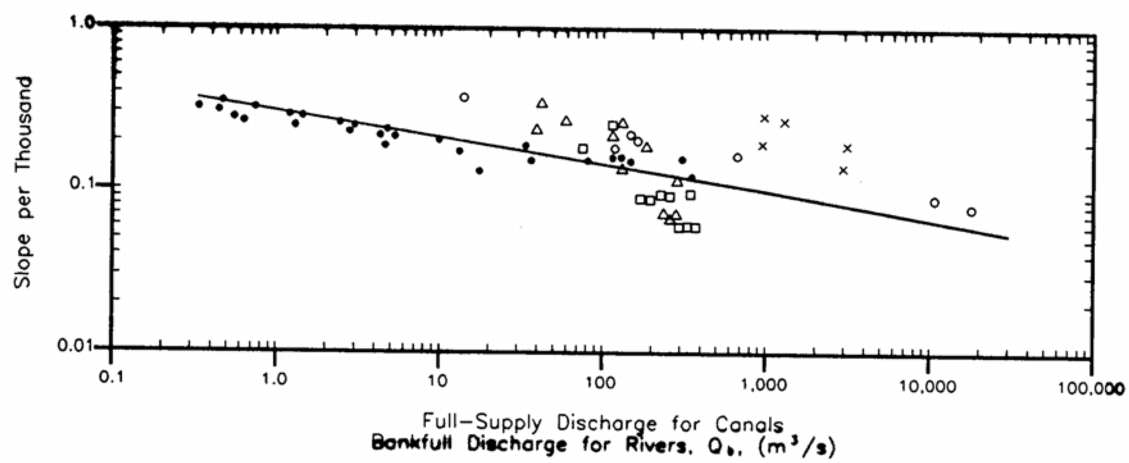
Subjective Criteria for Assignment of Channel Stability Categories (after Neill, 1973)

Source: (After Neill, 1973)

Figure 9.34b and c: Stable Channel Characteristics.



b) Depth versus Q



c) Slope versus Q

Figure 9.34d: Stable Channel Characteristics.

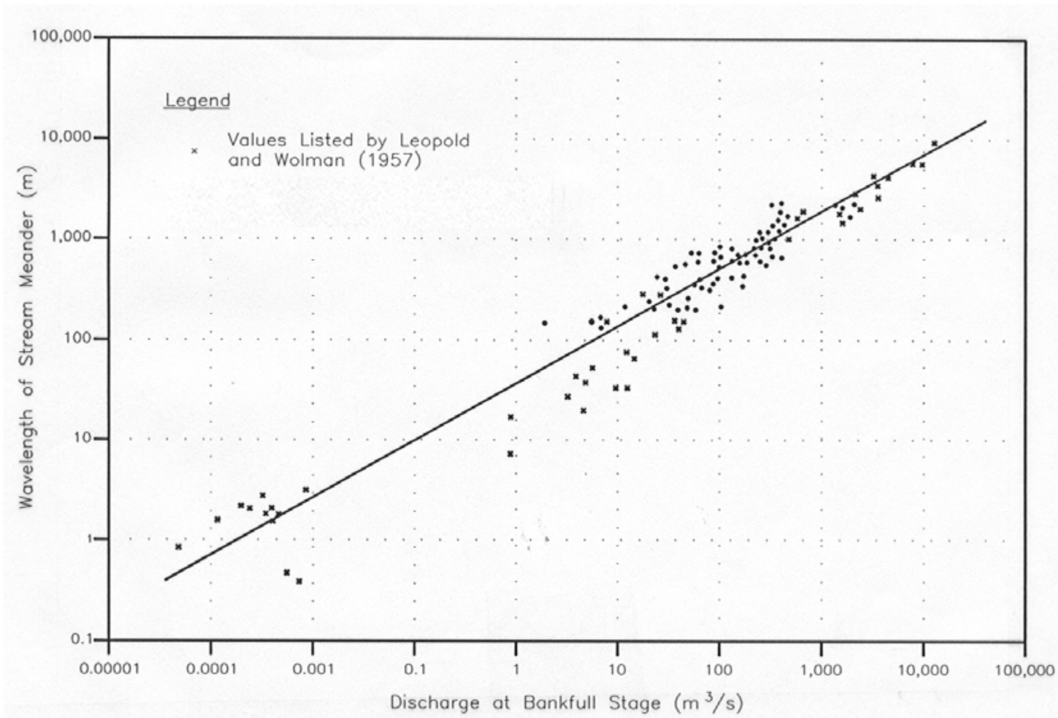
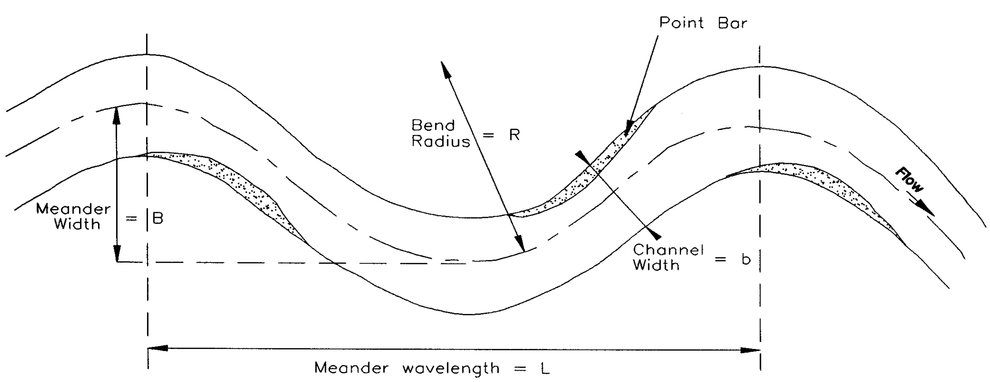


Figure 9.35: General Characteristics of Meandering Channels.



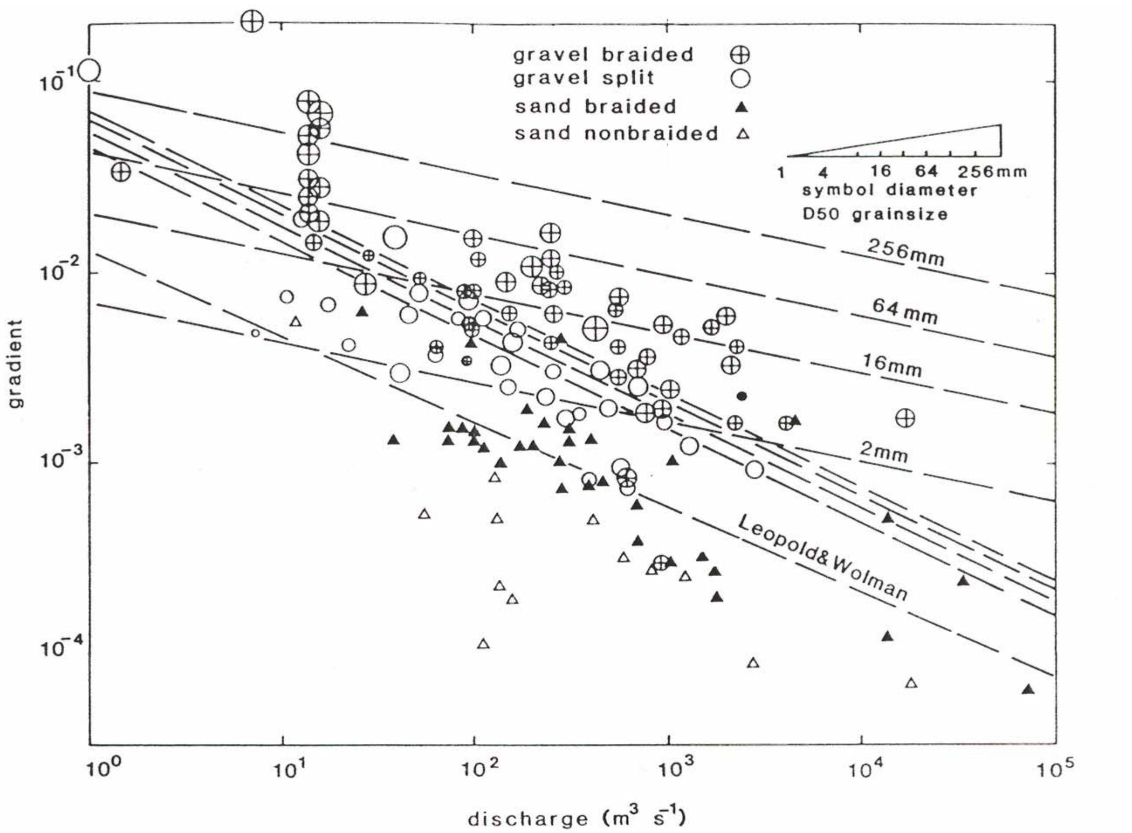
Meander Wavelength $L = 54\ Q_b^{\ 0.5}$

$L = 12.3\ b_{\ (mean)}$

Sinuosity = $\dfrac{\underline{\text{Thalweg Length}}}{\text{Valley Length}} > 1.5$

for meandering channels $b = 4.4\ Qb^{\ 0.5}$

Figure 9.36: Slope vs Discharge for Rivers.



Channel pattern in relation to slope, bankfull of 2-year discharge, and grain size showing tendency for fine-grained channels to braid at lower slopes. Data assembled by M. Church and previously published in Kellerhals (1982) and Ferguson (1984)

Source: (After Ferguson, 1987)

Figure 9.37: Pool and Riffle Sequence.

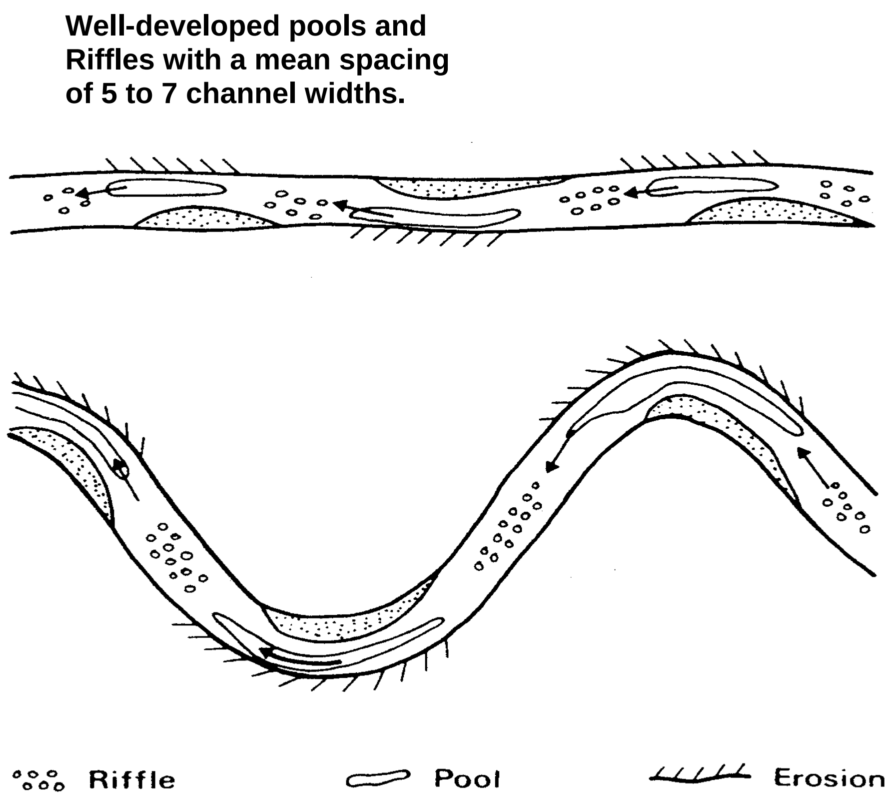
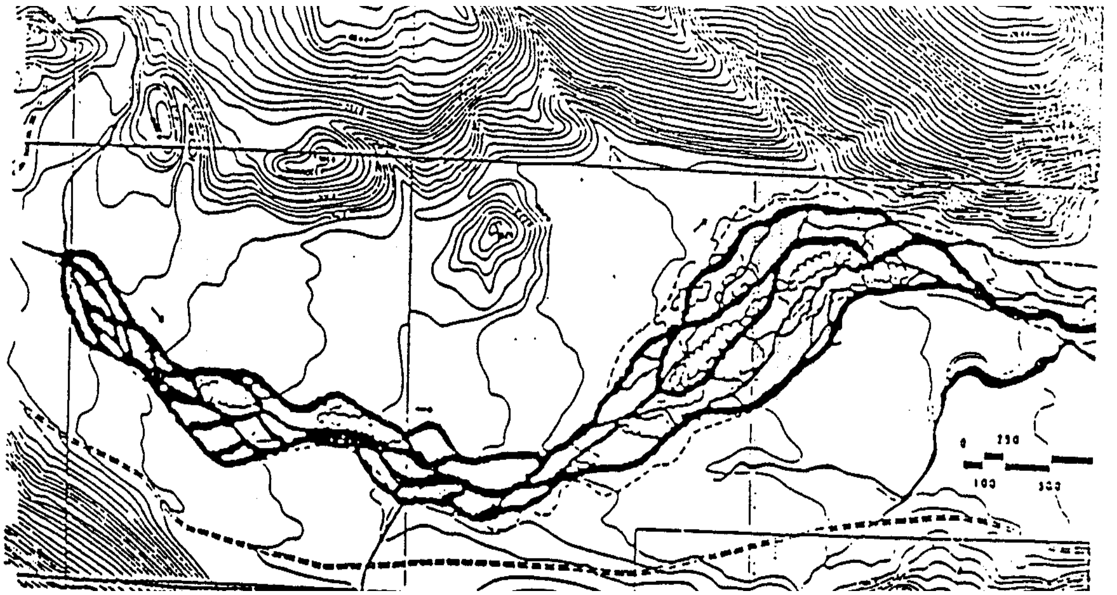
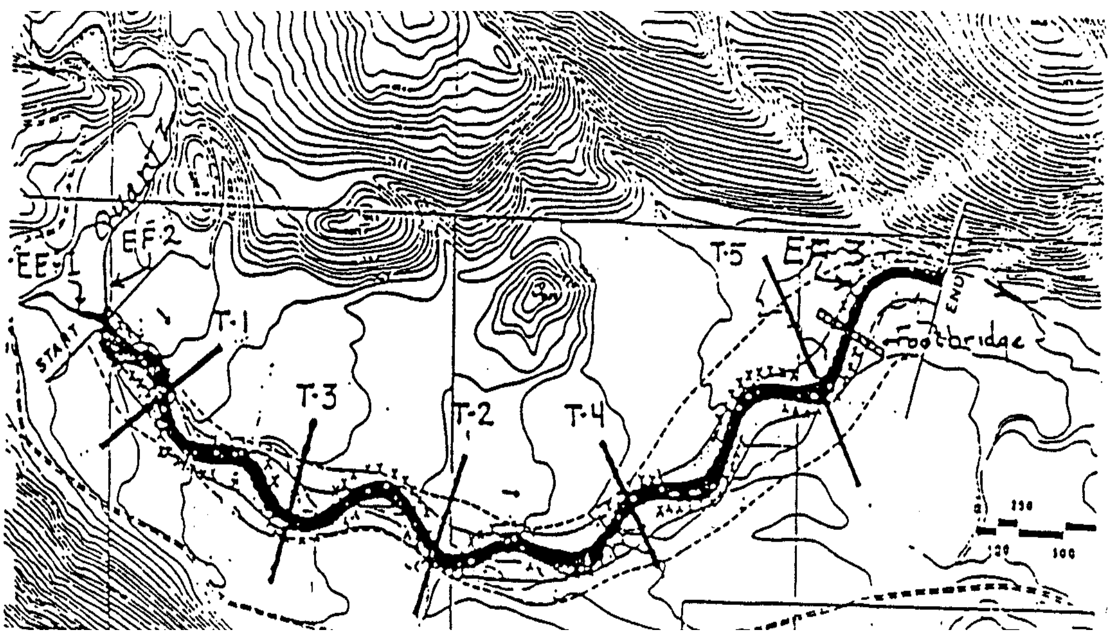


Figure 9.38: East Fork San Juan River in Colorado.



a) Braided Deposition Reach



b) New meandering system constructed in 1987

Source: (After Rosgen, 1988)

Figure 9.39: Differing Scenarios for Bed-Material Transport during Upstream and Downstream Progressing Degradation.

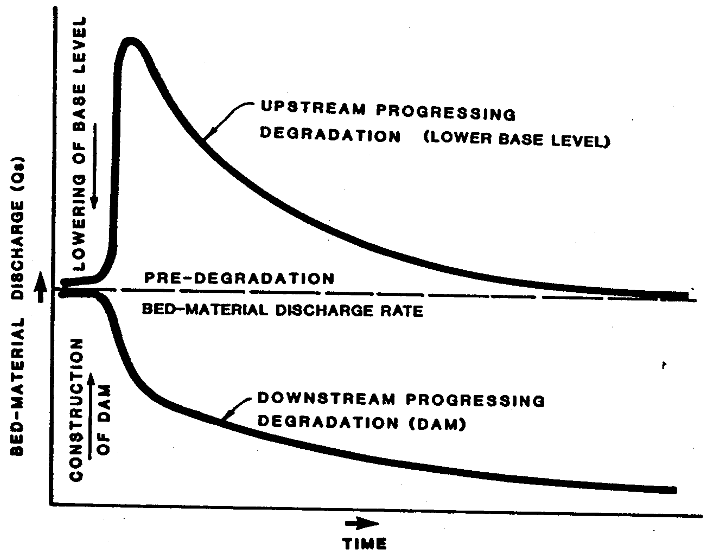


Figure 9.40: Specific Gauge Curve.

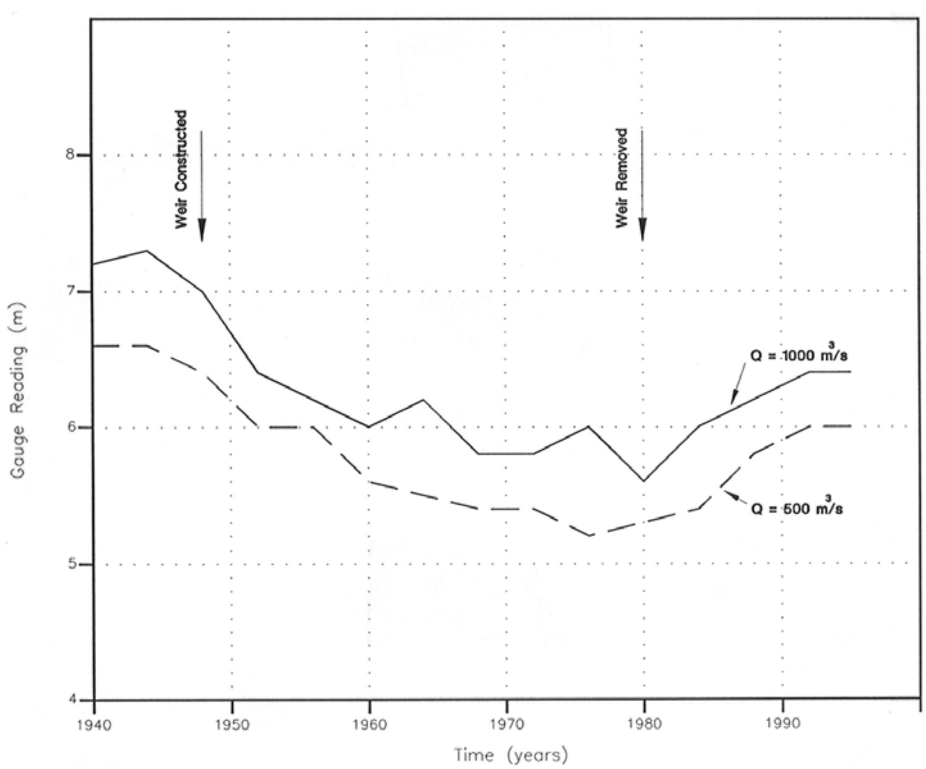
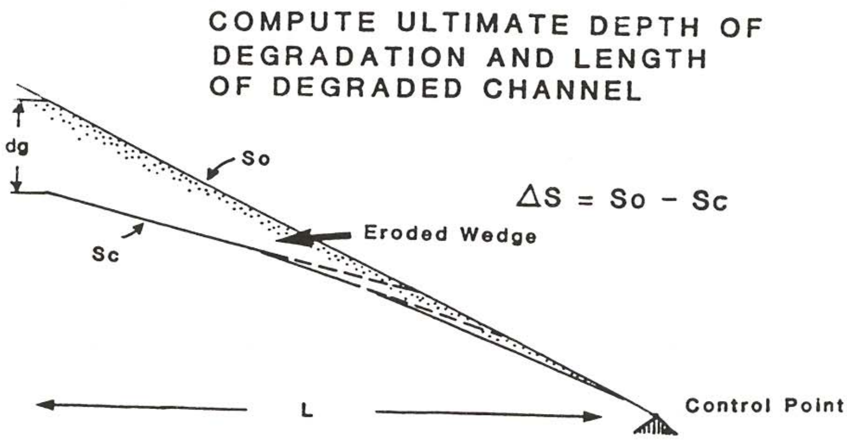
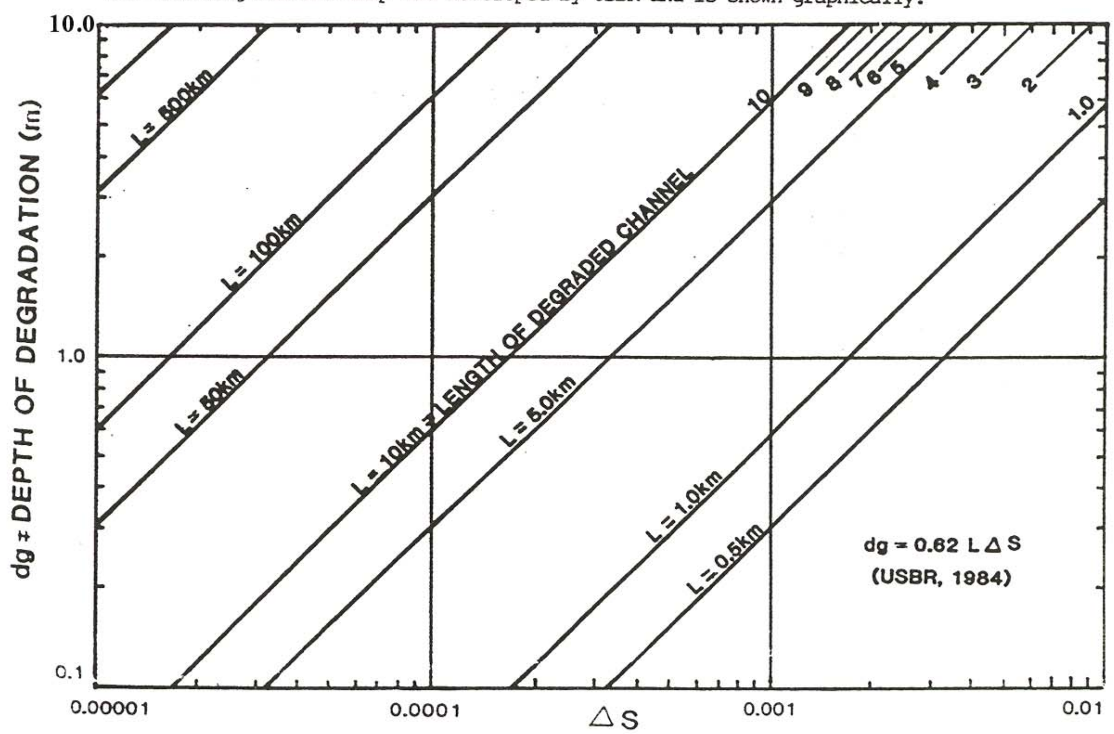


Figure 9.41: Stable Slope Method to Compute Degradation.



The following relationship was developed by USBR and is shown graphically.



Source: USBR, 1982.

Figure 9.42: Stable Channel Characteristics - Slope versus Bankfull Discharge.

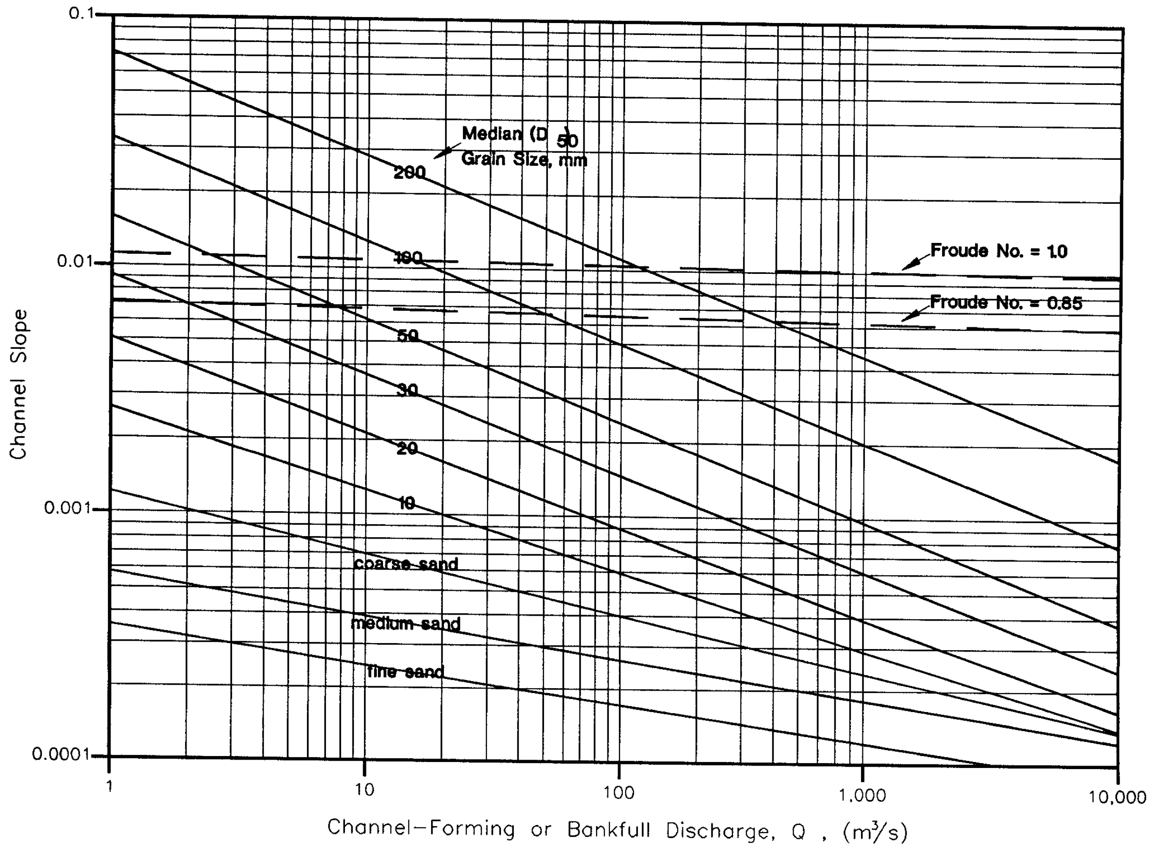
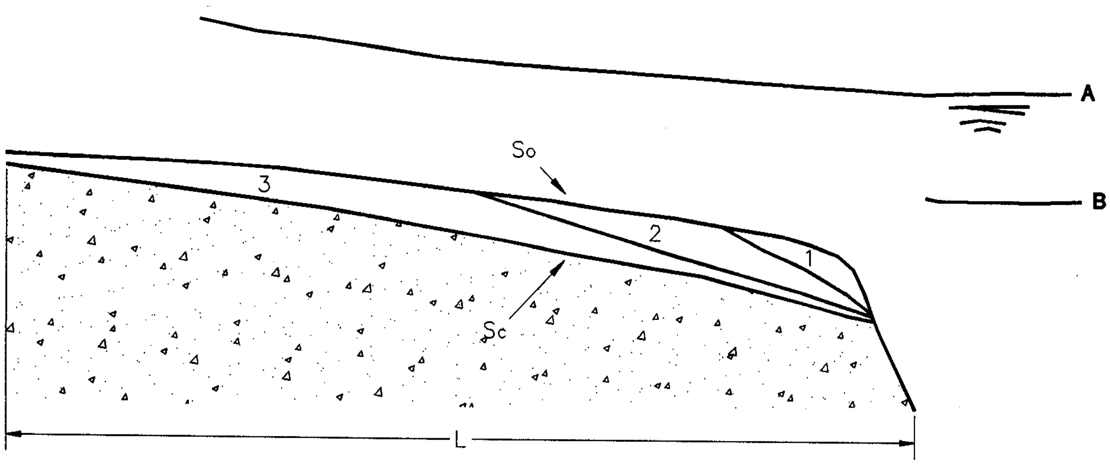
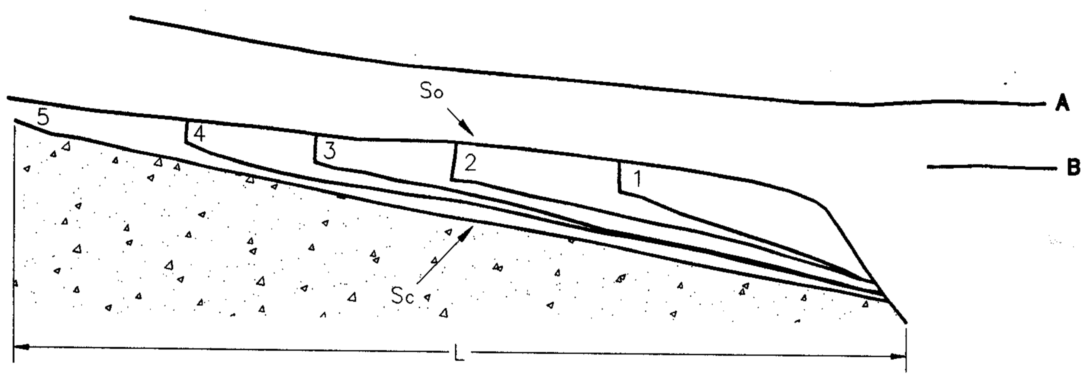


Figure 9.43: Upstream Progressing Degradation Processes.

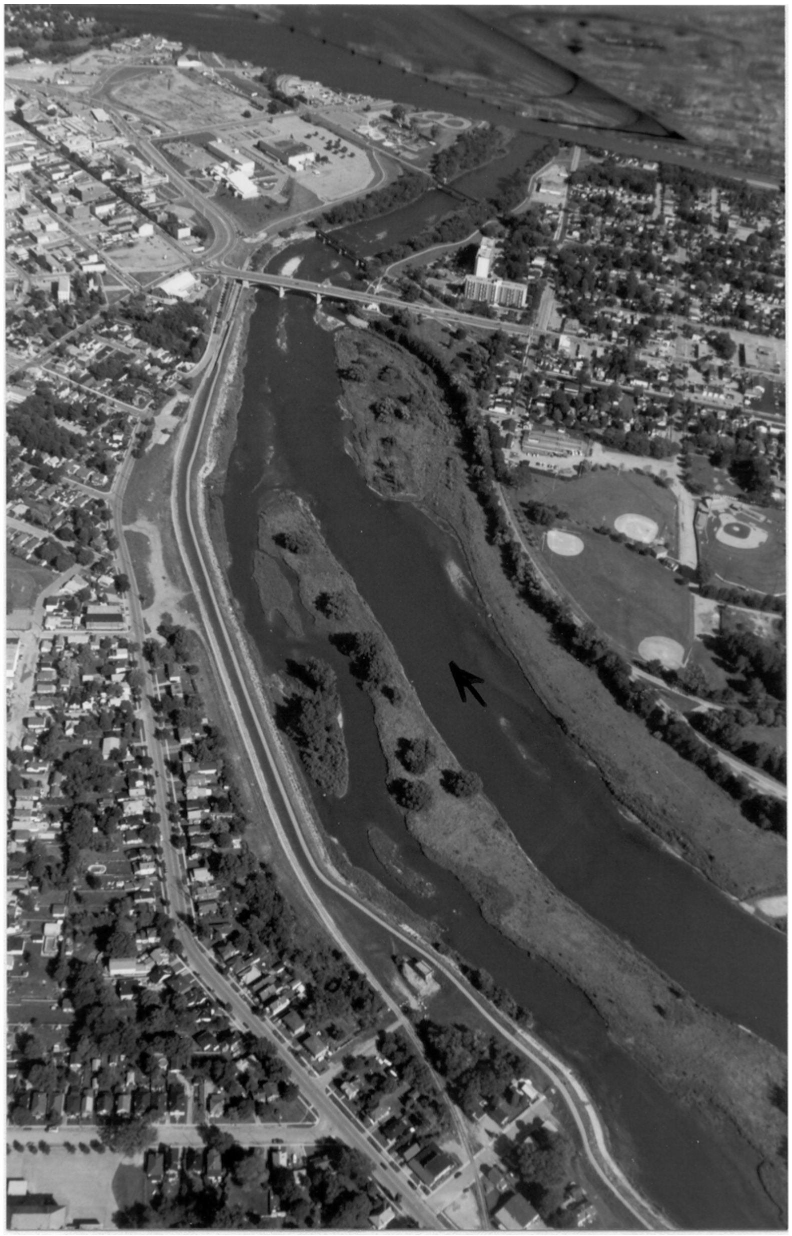


a) Rotation of River-Bed



b) Headcutting Process

Figure 9.44: Grand River, Brantford, Ontario.



The headcut channel is now located in the middle of the reach and the old channel is still evident along the left bank. The head cutting (upstream progressing degradation occurred after a navigation damn was removed just downstream from the highway bridge.

View is downstream, August 1994.

Figure 9.45: Deposition (Aggradation) along a Formerly Stable Stream.

The increase in sediment load was caused by diversion of street runoff into this stream. The logs and other fish habitat structures will eventually be covered by future sediment loads from upstream.

Figure 9.46: Erosion (Degradation) Occurring Upstream from the Deposition Reach Shown in the Last Figure (Figure 9.45).

The further degradation will result in bank slumping and removal of sand-gravel material from its bed. Habitat for fish will decline.

Figure 9.47: Types of Bars Found in Streams.

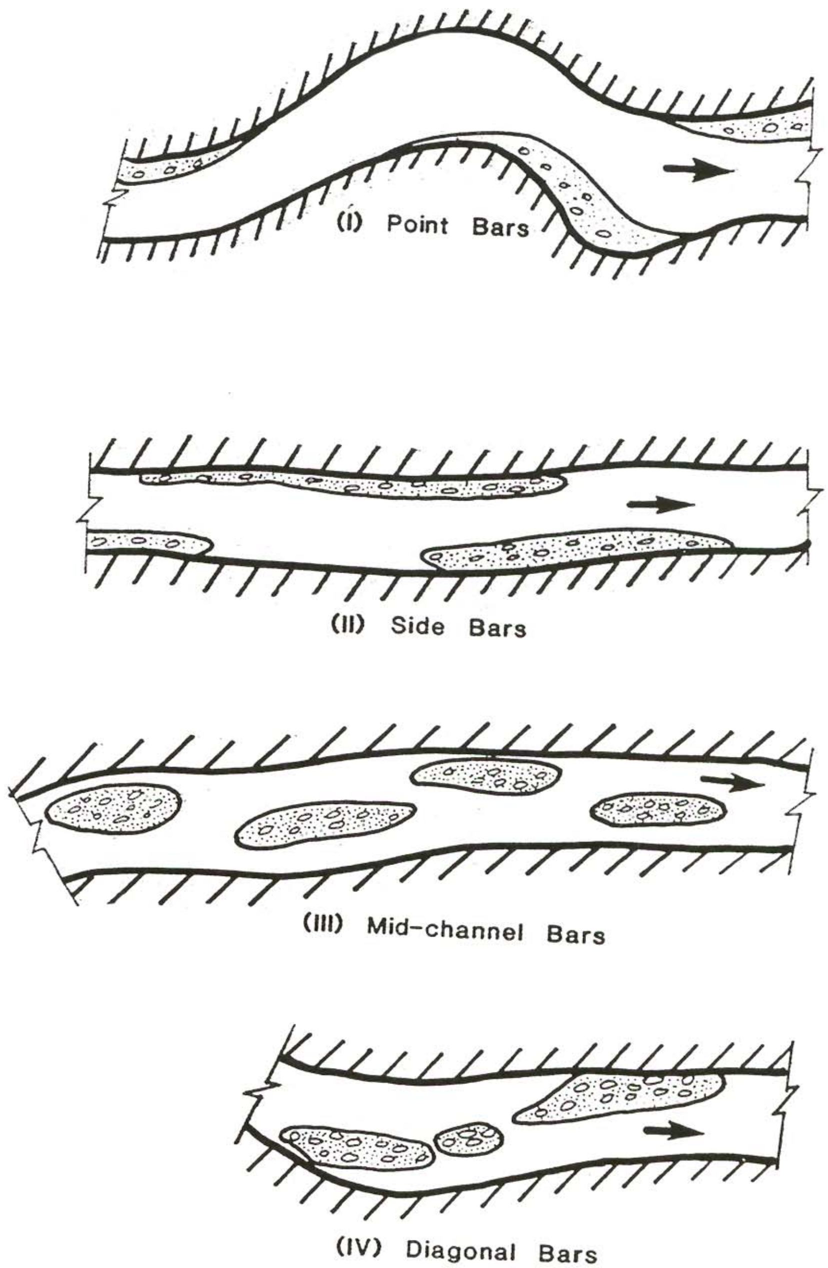
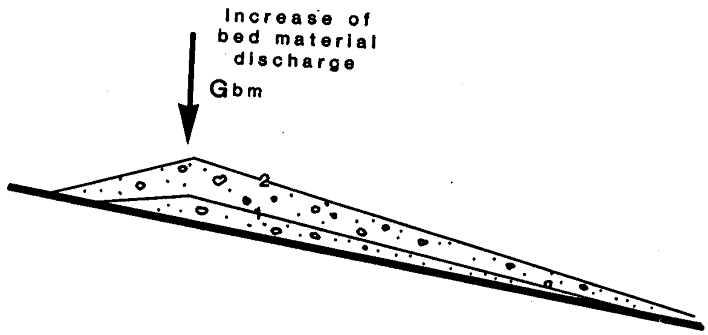


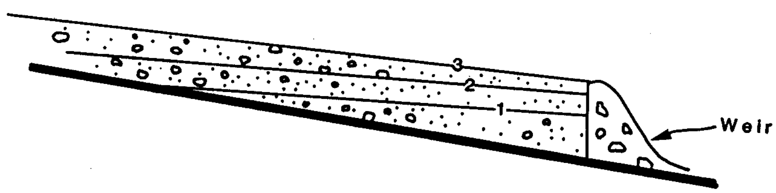
Figure 9.48: Deposition Indicated by Mid-channel Bars Upstream and Downstream from a Bridge.

Flow is from left to right. The bridge constriction results in backwater which causes reduced velocities upstream leading to deposition of sand and gravel.

Figure 9.49: Deposition Scenarios.



1. DOWSTREAM PROGRESSING AGGRADATION



2. UPSTREAM PROGRESSING AGGRADATION

Figure 9.50: Grand River at Caledonia, Ontario.

Relatively low navigation dam just downstream from railway bridge. Deposition of bed material has probably occurred above the dam but this should pose no problem to the bridge since it has a high deck level.

August 1994

Figure 9.51: Excavation of Trench across a Small Stream for a Sewer Line.

Upstream from the line, a bridge will be constructed. The stream is actively undergoing aggradation due to increase of bed-material load (sand gravel) from upstream gravel pits.

Flow is left to right.

Figure 9.52: Factors Affecting Lateral Shifting (Channel Stability).

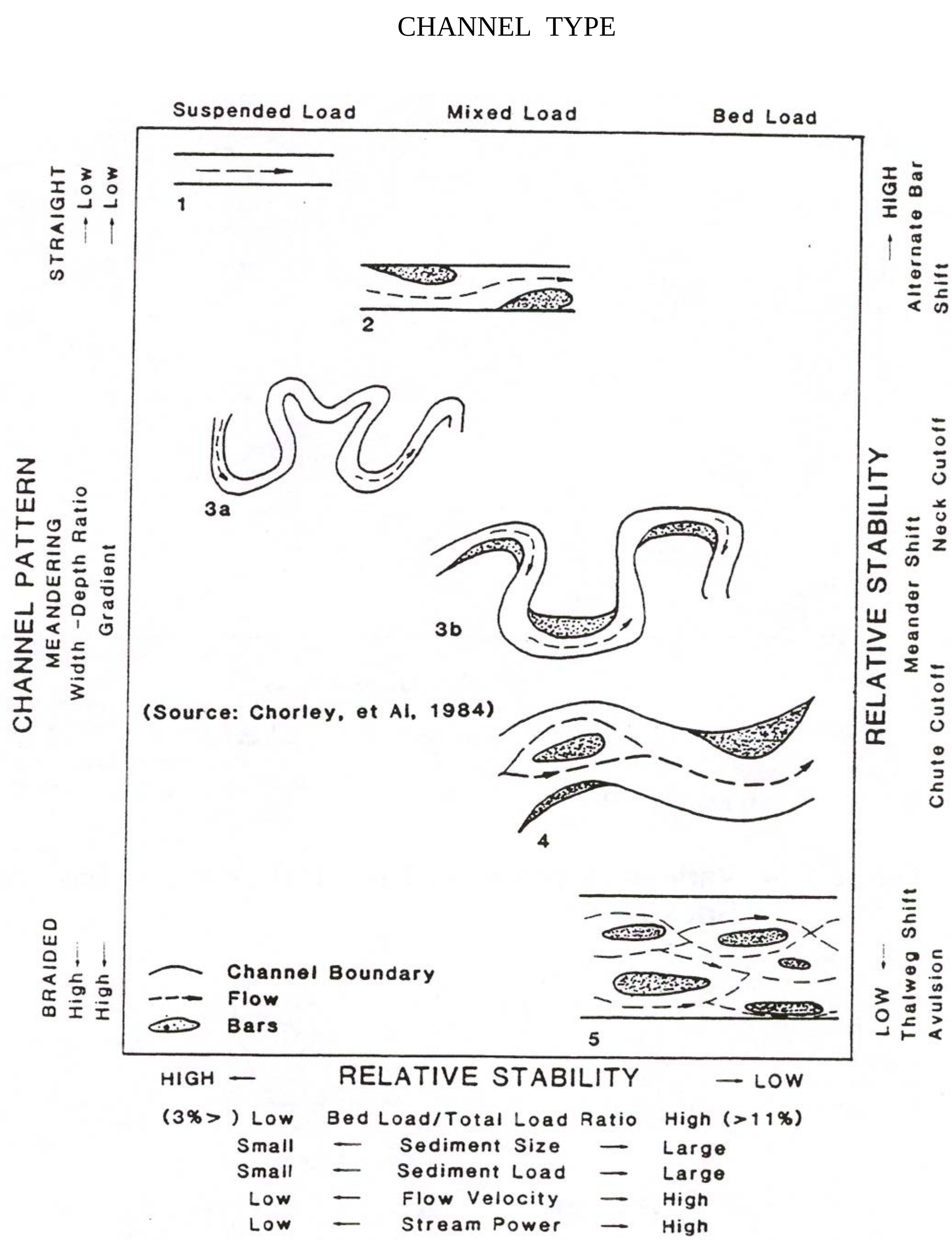
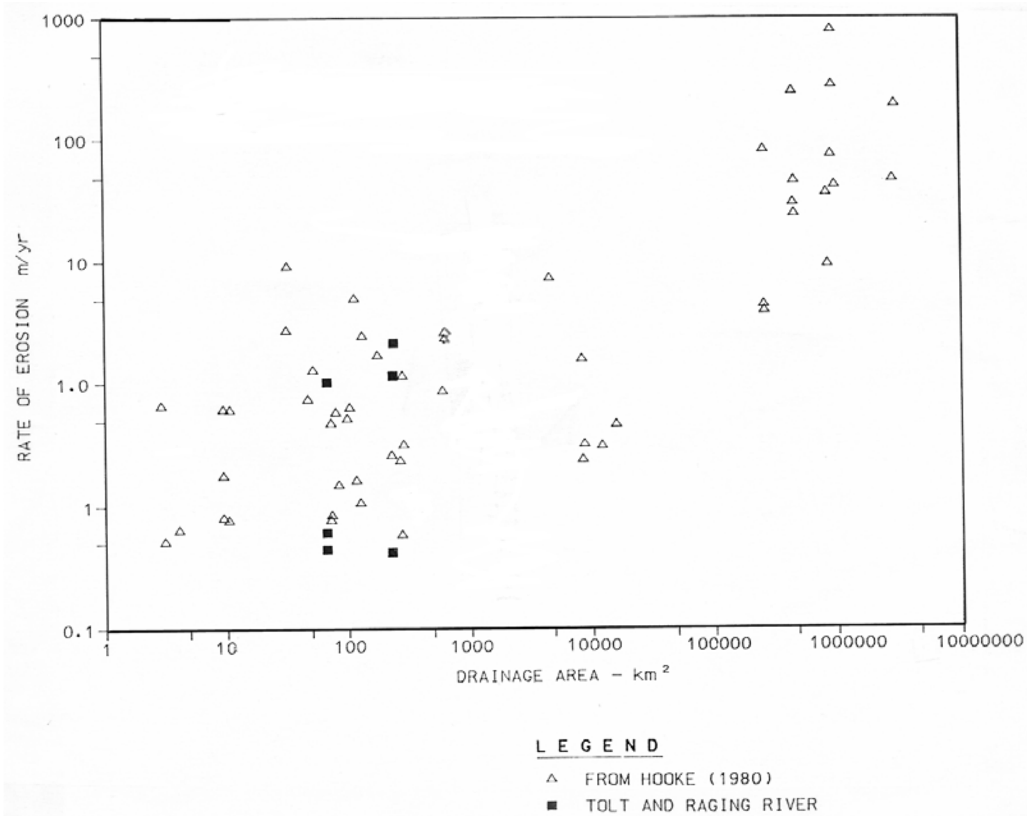


Figure 9.53: Rates of Annual Bank Erosion Versus Drainage Area.



Source: (From Hooke, 1980)

Figure 9.54: Various Movements of Meander Loops as Developed by Hooke (1977).

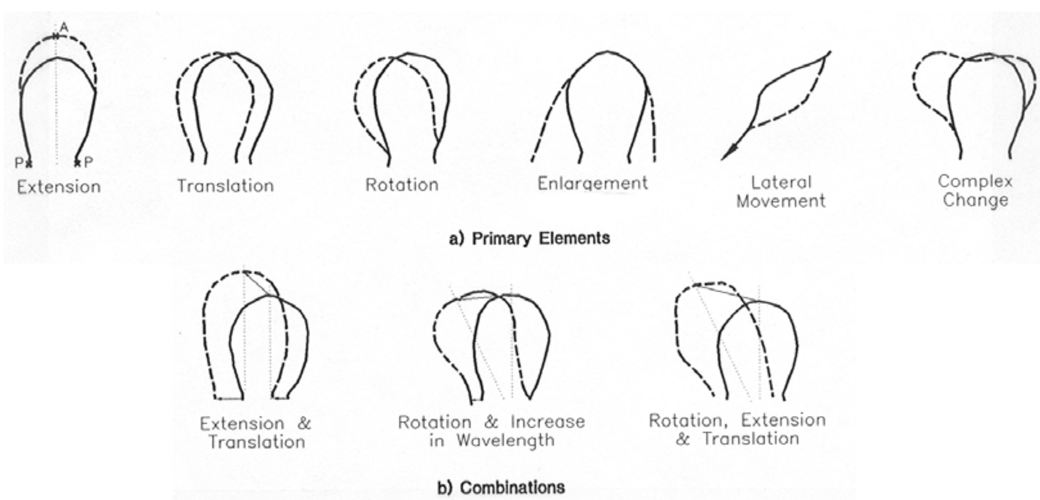


Figure 9.55: Progressive Translation of Meander Bend.

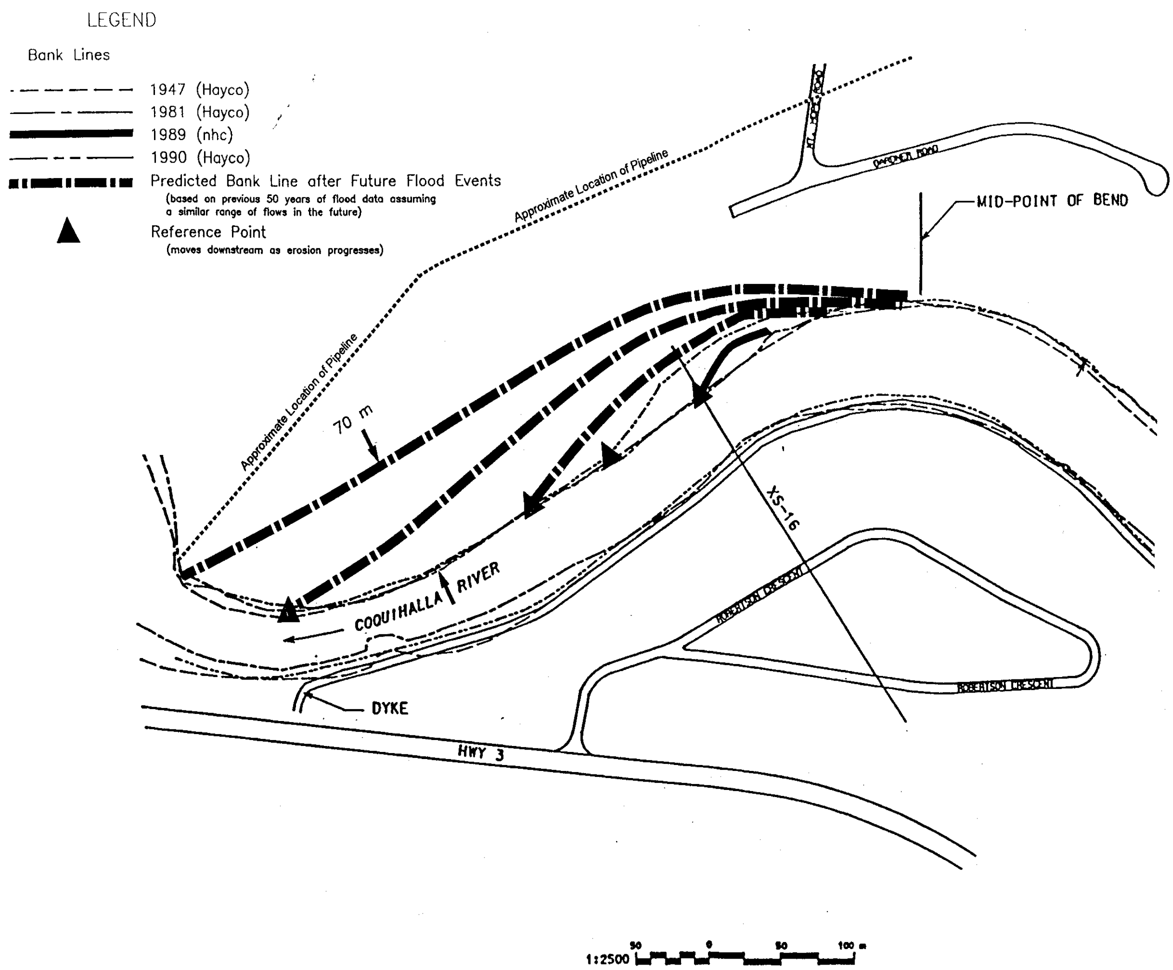


Figure 9.56: Nith River, Ontario.

Figure 9.57: Hydrographs Showing Floods.

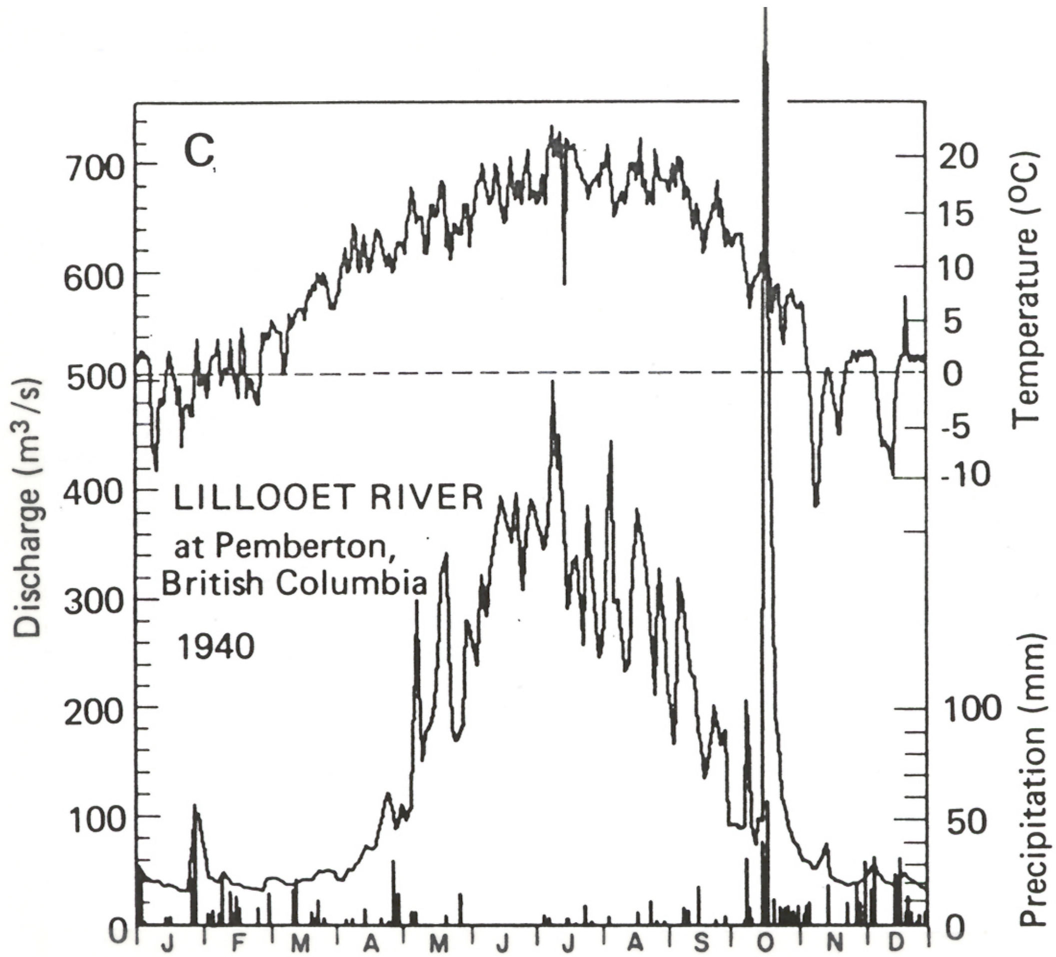


Figure 9.58: Frequency Curve.

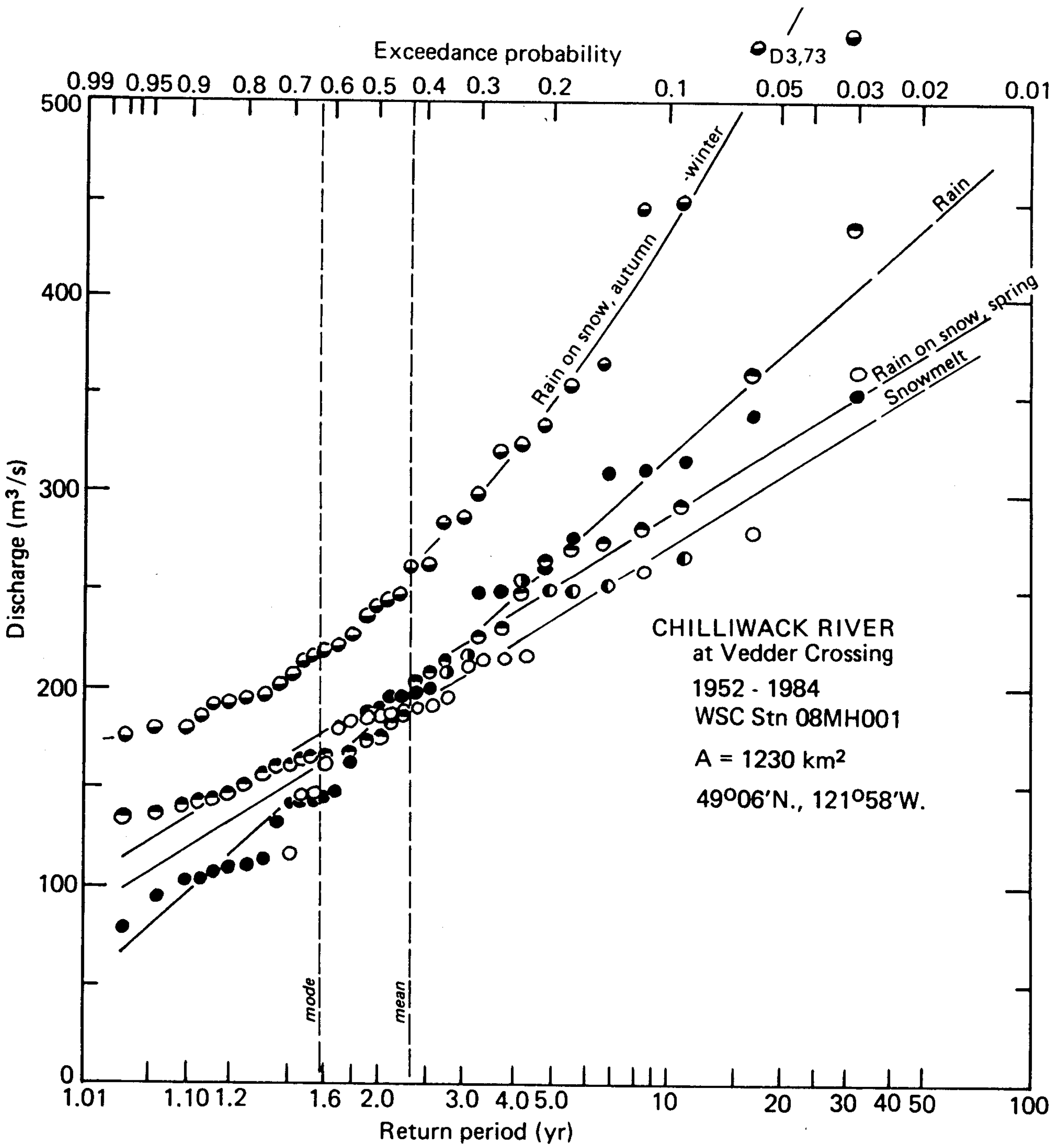


Figure 9.59: Flood Zones in Southern Ontario.

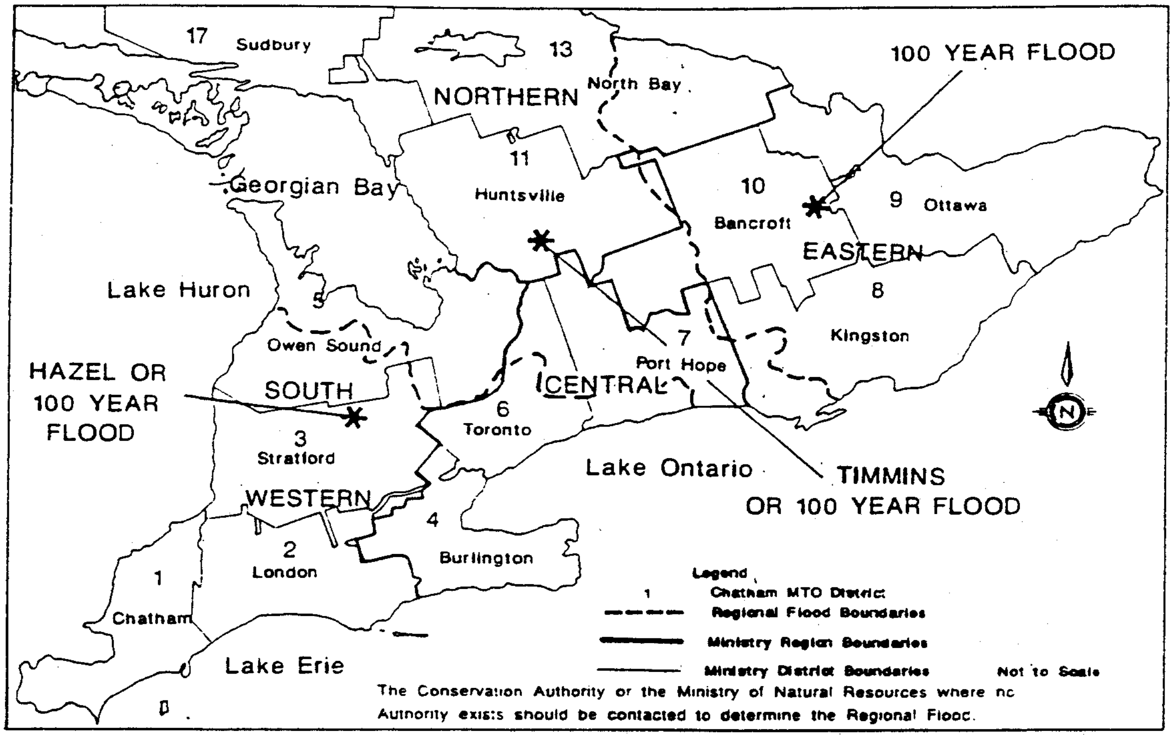
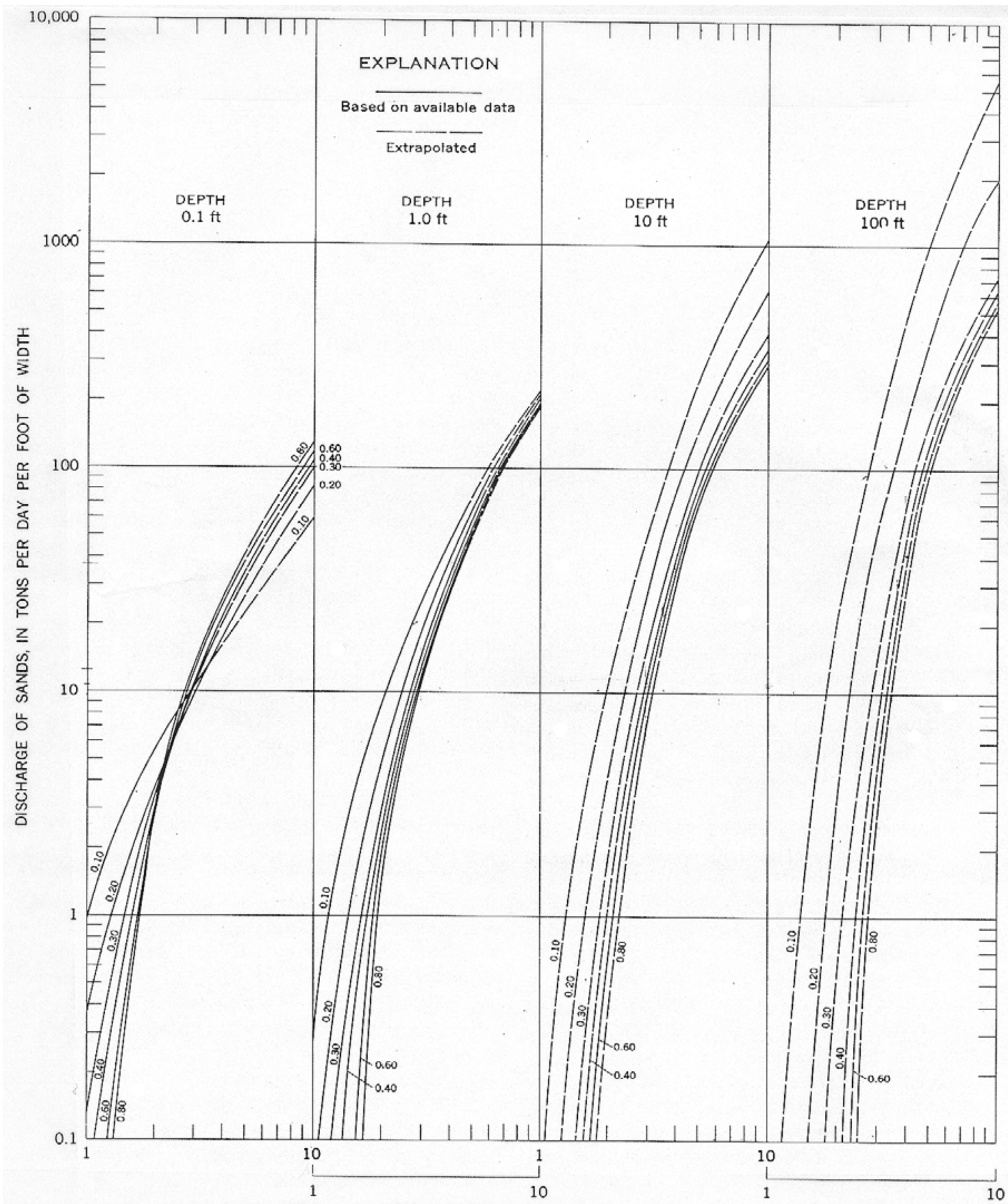


Figure 9.60: Colby Chart for Bed-Material Load in Sand-Bed Rivers.



MEAN VELOCITY, IN FEET PER SECOND

Note: numbers adjacent to curves indicate median diameter of bed material in mm.

Figure 9.61: Shields Diagram for Initiation of Motion Particles.

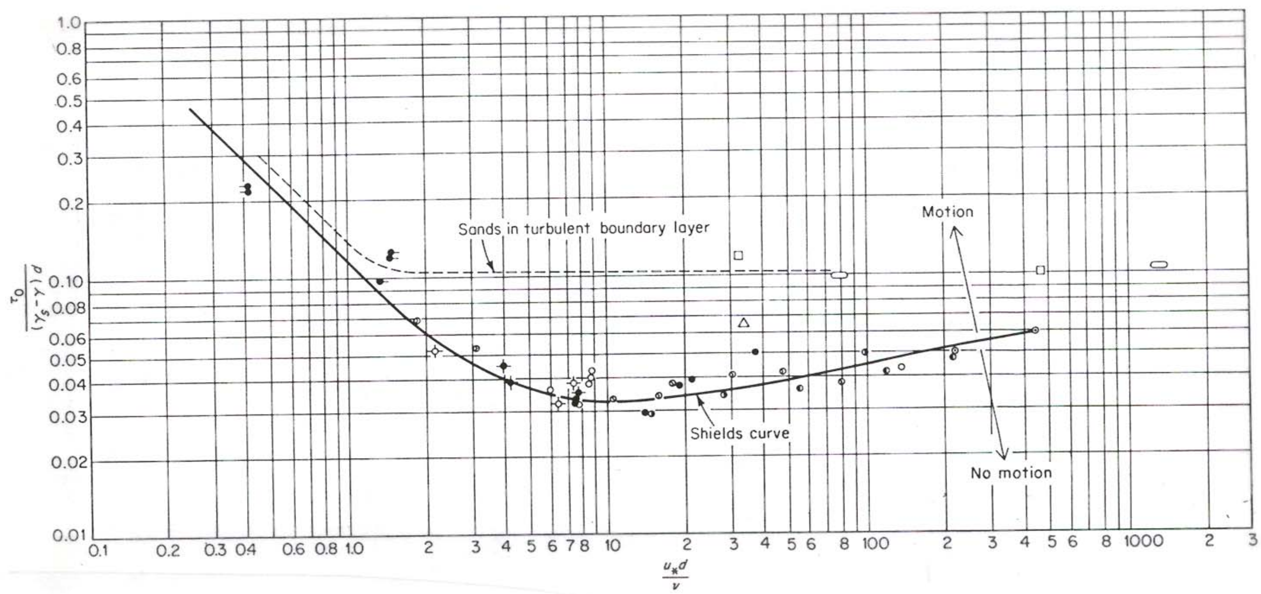
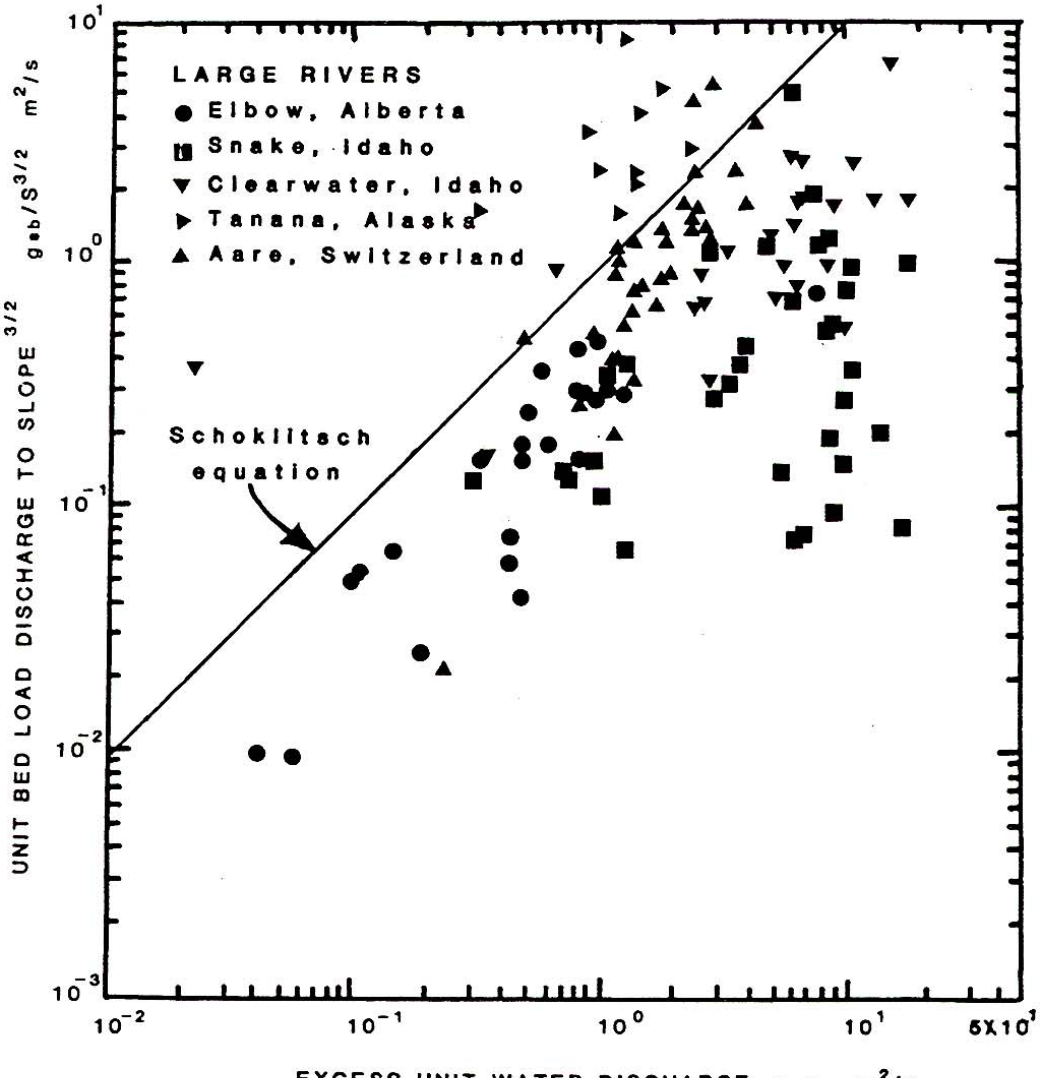


Figure 9.62: Chart for Gravel Transport.

NOTE: FOR A SPECIFIC REACH OF RIVER DETERMINE SLOPE, D_{40} OF BED MATERIAL AND CROSS-SECTION PROPERTIES.

THEN DETERMINE CRITICAL UNIT DISCHARGE FROM CHART a., AND GRAVEL TRANSPORT, FOR VARIOUS FLOWS, FROM CHART b.



(After Bathurst, et al, 1985)

b. SCHOKLITSCH EQUATION

Figure 9.63: Schoklitsch Chart for Initiation of Motion of Particles on Stream Bed.

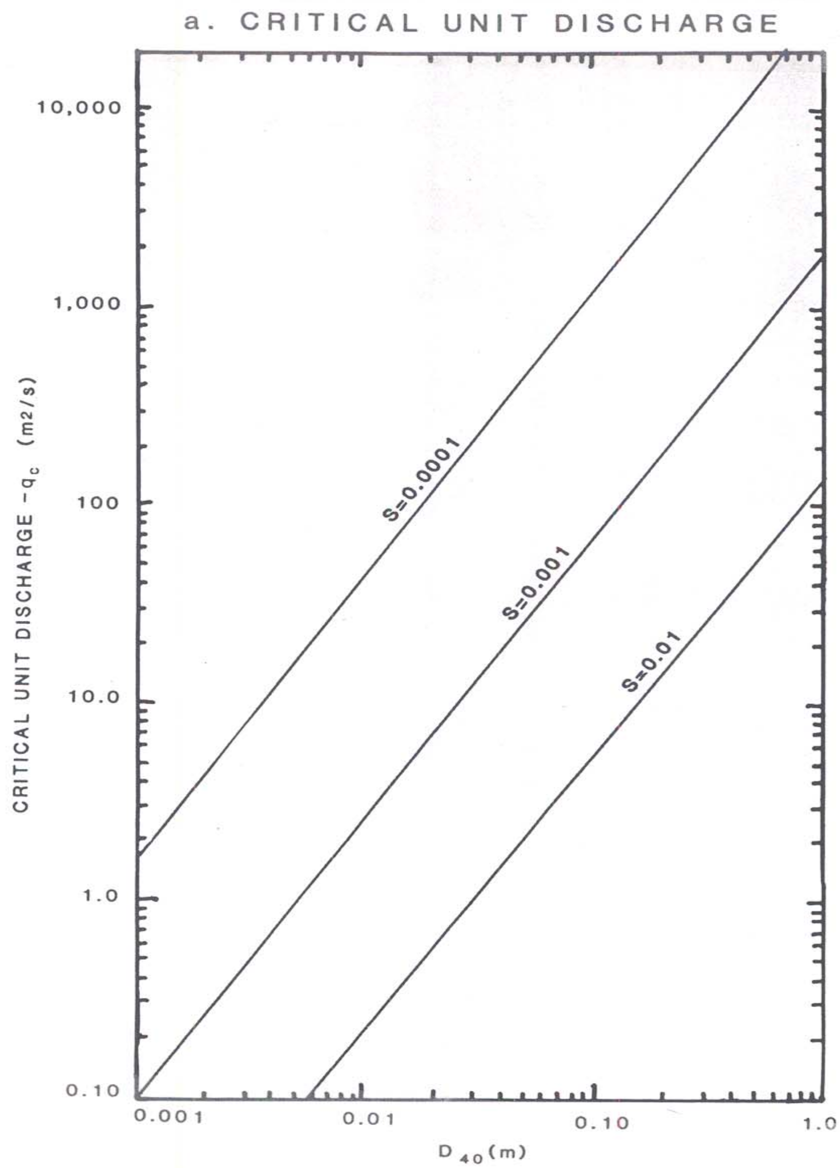


Figure 9.64: Suspended Sediment Loads for Ontario Streams.

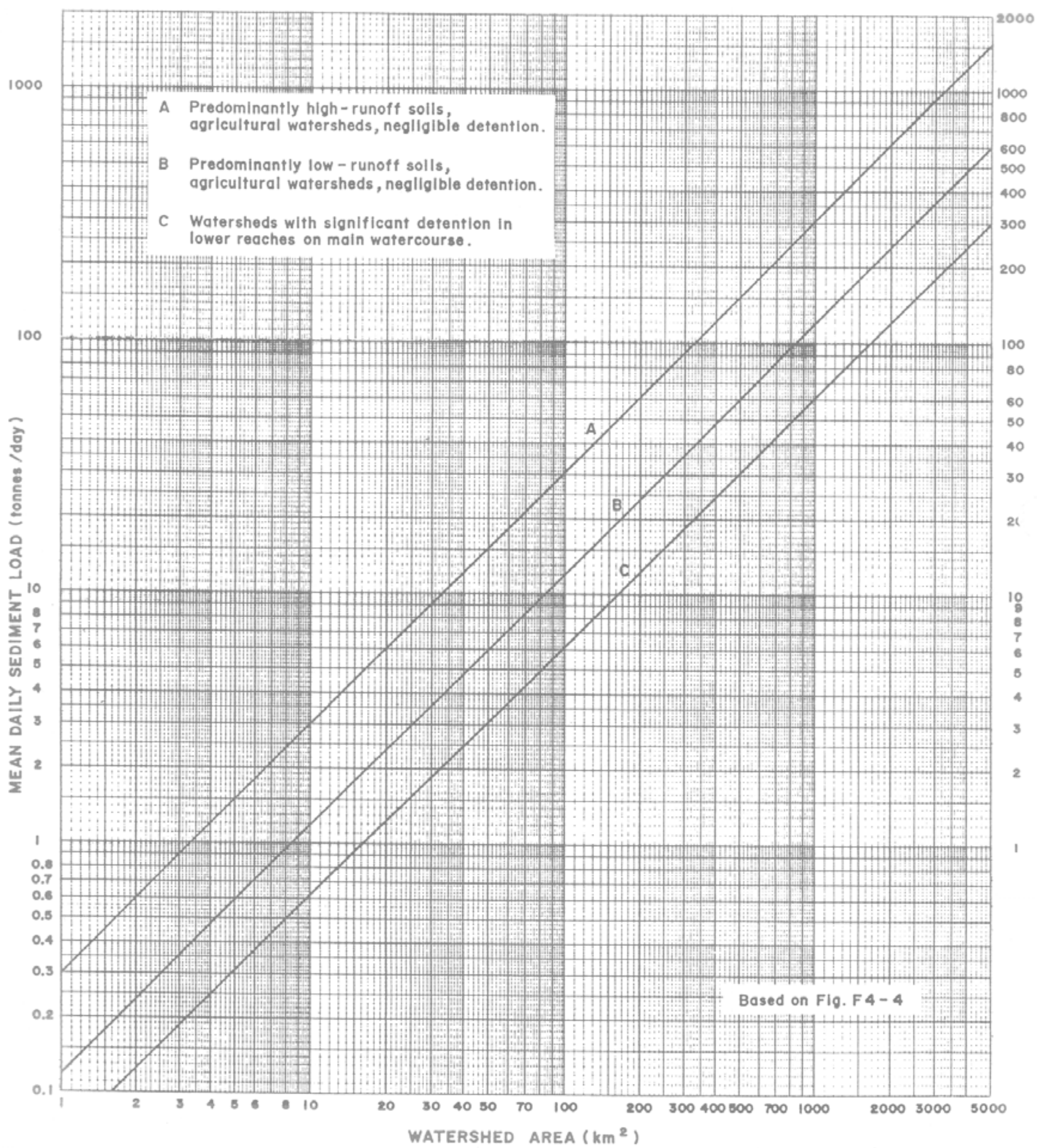


Figure 9.65: Topographic Map of Gravel River.

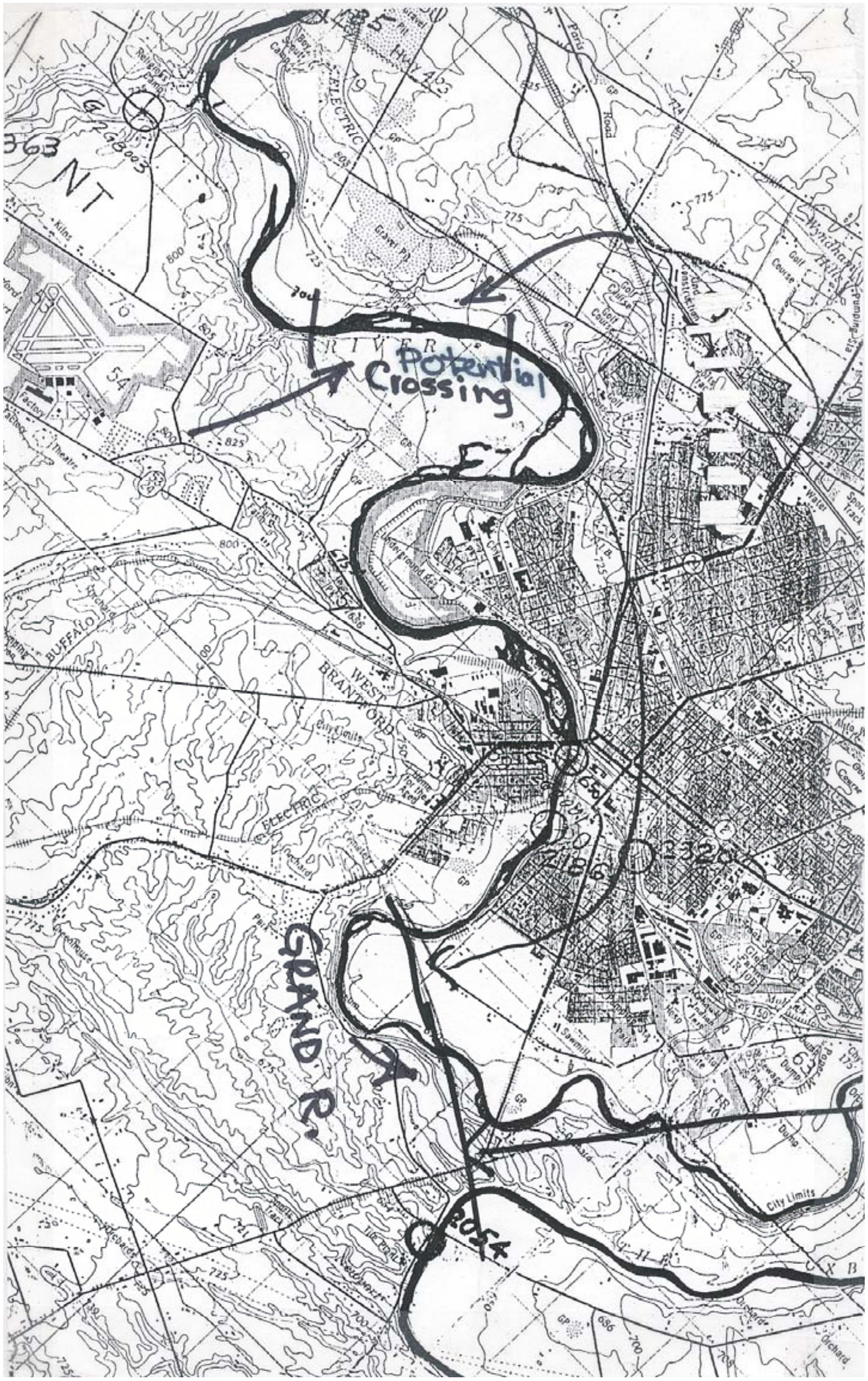


Figure 9.66: Air Photos of Grand River.

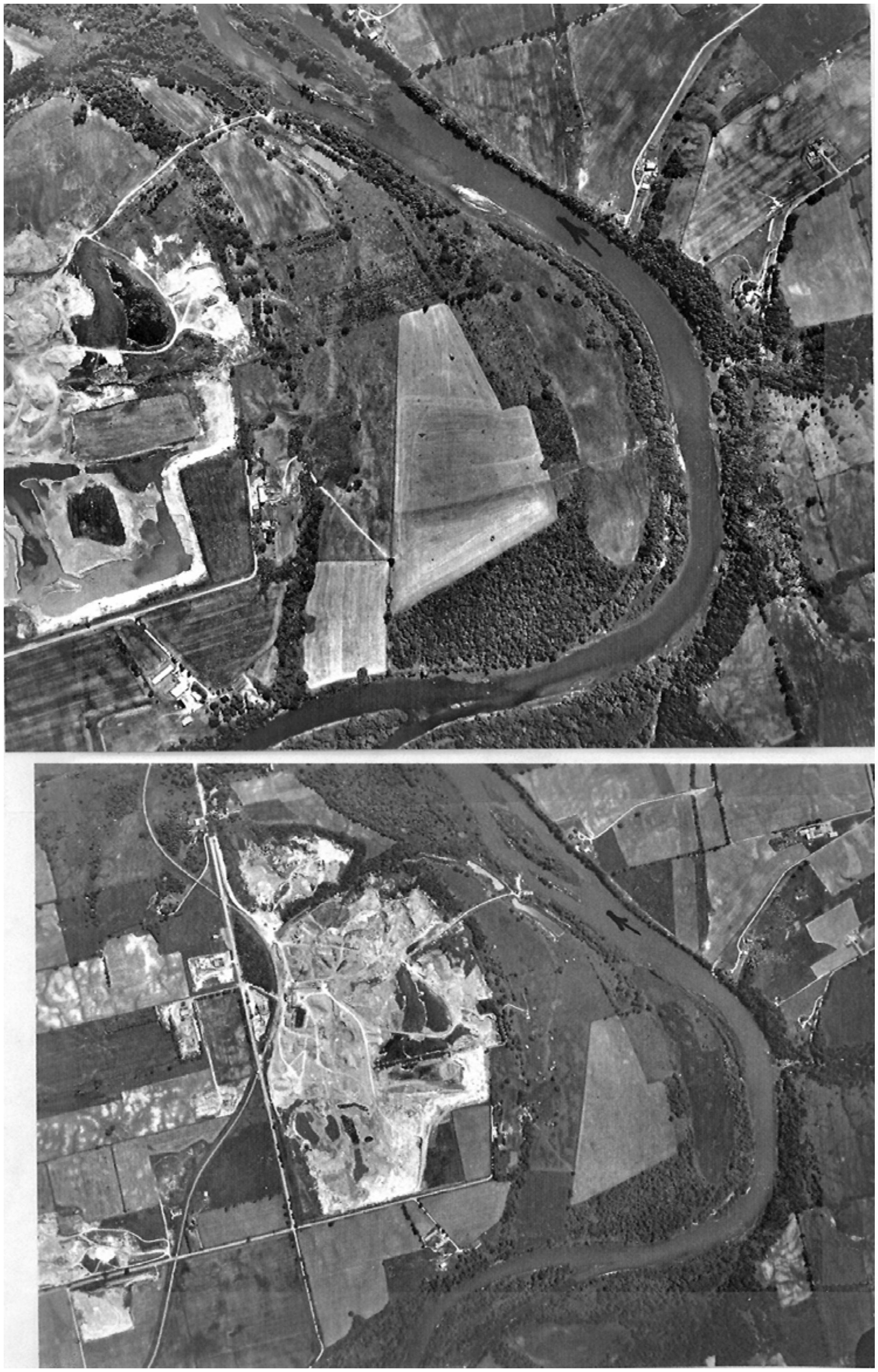
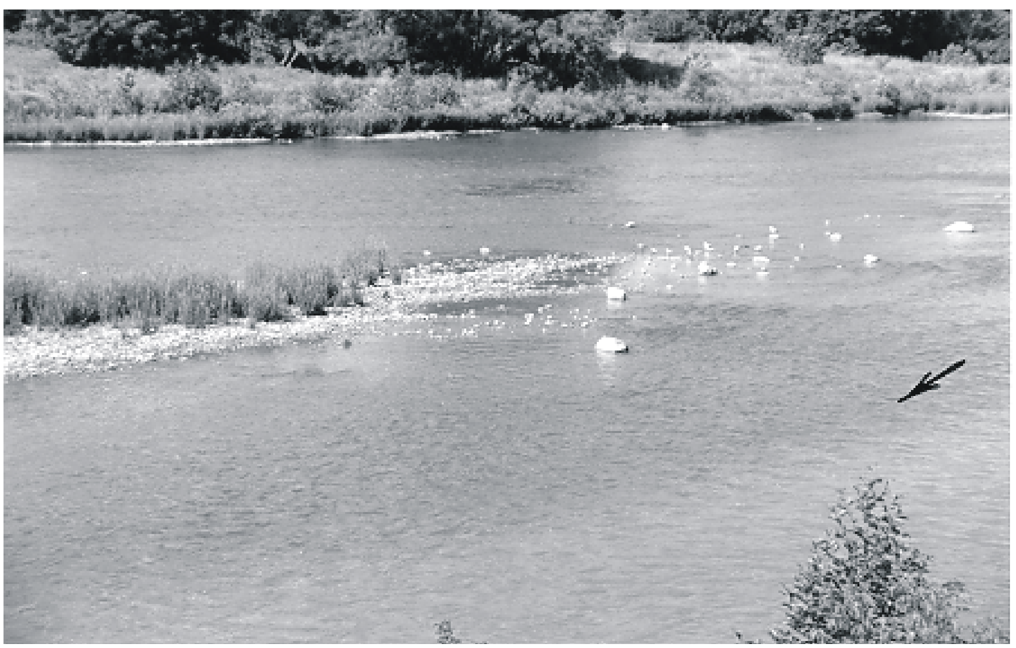


Figure 9.67: Grand River - Problem.

Photo of proposed bridge crossing site.
View is upstream.

Figure 9.68: Grand River - Problem.



Bed-Material along river downstream from site.

Figure 9.69a: River Data Sheet No. Geog1

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DEPARTMENT OF CIVIL ENGINEERING

RIVER DATA SHEET No. Geog. 1/71

GEOGRAPHIC FEATURES

Reach Name: GRAND R.—Brantford Reach No: ______ Date of Analysis: ______ Analysis By: ______

Scale of Air Photos: ______ Scale of Map: 1:50,000

NOTE: Complete codes by circling the appropriate number(s). Use "-1" for "unknown" and "0" for "not applicable".

General Description of the Terrain in the Vicinity of the Surveyed Reach, above Valley

Terrain:
1 mountainous
2 foothills
3 uplands
4 hills
5 plains
6 lowlands

Vegetation:
0 0 0 not applicable
1 1 1 almost none
2 2 2 grass
3 3 3 shrubs
4 4 4 sparsely forested, 0-25%
5 5 5 moderately forested, 25-75%
6 6 6 heavily forested, 75-100%
7 7 7 swamp or muskeg

Forest type:
0 0 0 not applicable
1 1 1 deciduous
2 2 2 coniferous

Comments: ______

Land use:
0 0 no cultivation or built-up area
1 1 partly cultivated
2 2 mainly cultivated
3 3 partly built-up
4 4 urbanized

Surficial geology:
1 1 1 bedrock
2 2 2 ground moraine
3 3 3 hummocky moraine
4 4 4 lacustrine deposits
5 5 5 glacio-fluvial dep.
6 6 6 fluvial deposits
7 7 7 aeolian deposits

Valley Characteristics above Valley Flat

Valley measurements:
✓ within reach
___ within reach and immediate vicinity
depth: 100 ft.
top width: varies mi.
bottom width: ______ mi.

Slumping of valley walls:
0 none
1 occasional
2 frequent

Length of reach with slumping valley walls (contact length in percent of total length of banks): —

Vegetation on valley wall:
0 0 not applicable
1 1 almost none
2 2 grass
3 3 shrubs
4 4 sparsely forested
5 5 moderately forested
6 6 heavily forested
7 7 swamp or muskeg

Forest type on valley wall:
0 0 not applicable
1 1 deciduous
2 2 coniferous

Comments: ______

Terraces

Terrace presence:
0 none
1 indefinite
2 fragmentary
3 continuous

Number of levels:
0 not applicable
1 one level
2 two levels
_ ___ levels
9 several levels

Comments (in particular land use and vegetation): ______

Relation of Channel to Valley

Valley type:
0 not applicable
1 stream cut valley
2 stream cut valley in wide valley
3 wide mountainous valley

If no valley:
0 valley present
1 on alluvial fan
2 on alluvial plain
3 in delta
4 in old lake

Underfit:
0 not applicable or not obviously underfit
1 obviously underfit

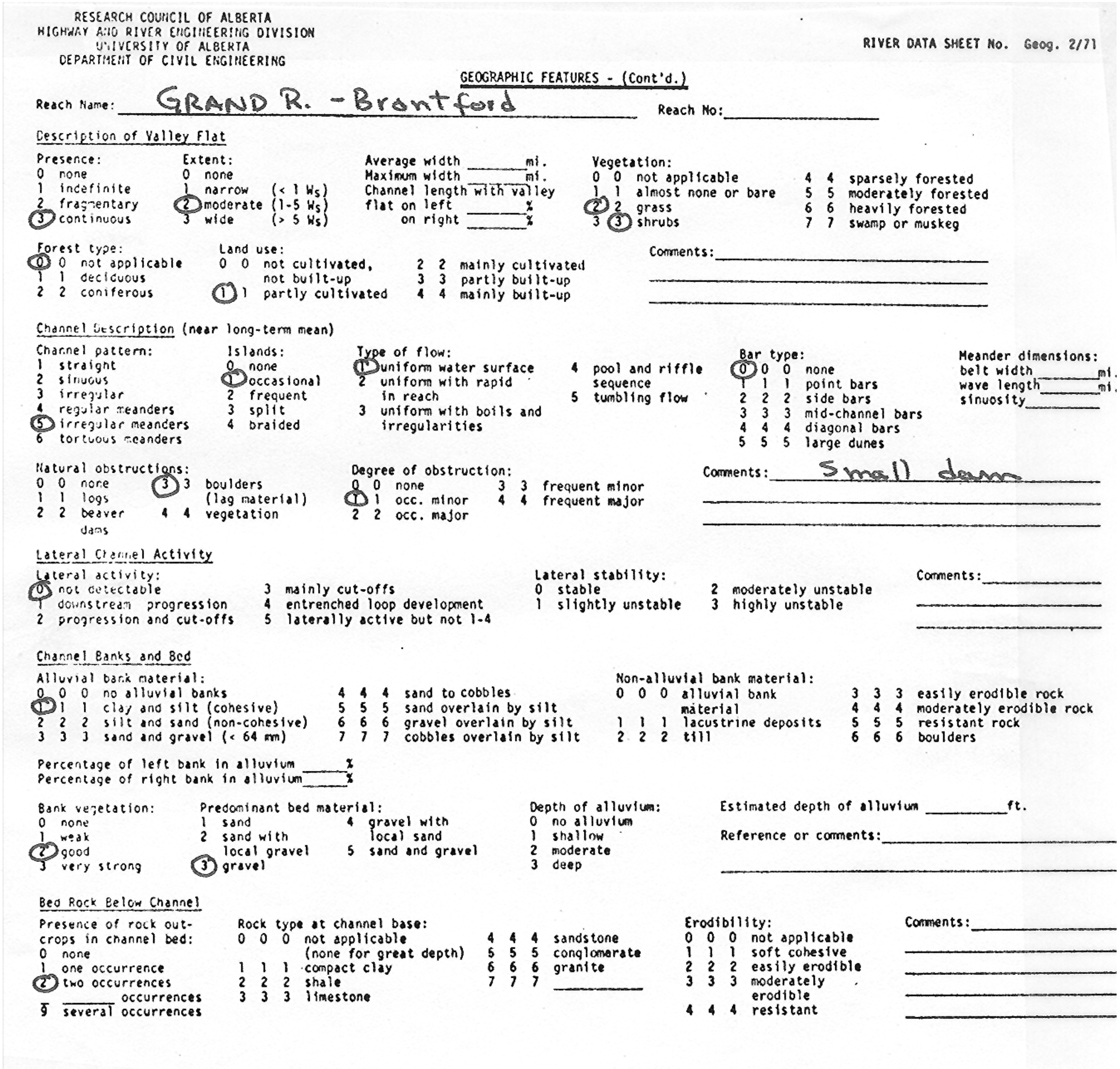
Local lateral constriction:
0 none
1 one
2 two
___ cases
9 several cases

Relation of channel to valley bottom (vertical):
0 not applicable
1 not obviously degrading or aggrading
2 partly entrenched
3 entrenched
4 aggrading

Relation of channel to valley walls or to high, resistant terraces (lateral):
0 not applicable (no valley or free)
1 occasionally confined
2 frequently confined
3 confined
4 entrenched

Comments: ______

Figure 9.69b: River Data Sheet No. Geog2



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RIVER DATA SHEET No. Geog. 2/71

GEOGRAPHIC FEATURES - (Cont'd.)

Reach Name: GRAND R. - Brantford   Reach No: ______

Description of Valley Flat

Presence:
0 none
1 indefinite
2 fragmentary
3 continuous

Extent:
0 none
1 narrow (< 1 Ws)
2 moderate (1-5 Ws)
3 wide (> 5 Ws)

Average width ______ mi.
Maximum width ______ mi.
Channel length with valley flat on left ______ %
on right ______ %

Vegetation:
0 0 not applicable
1 1 almost none or bare
2 2 grass
3 3 shrubs
4 4 sparsely forested
5 5 moderately forested
6 6 heavily forested
7 7 swamp or muskeg

Forest type:
0 0 not applicable
1 1 deciduous
2 2 coniferous

Land use:
0 0 not cultivated, not built-up
1 1 partly cultivated
2 2 mainly cultivated
3 3 partly built-up
4 4 mainly built-up

Comments: ______

Channel Description (near long-term mean)

Channel pattern:
1 straight
2 sinuous
3 irregular
4 regular meanders
5 irregular meanders
6 tortuous meanders

Islands:
0 none
1 occasional
2 frequent
3 split
4 braided

Type of flow:
1 uniform water surface
2 uniform with rapid in reach
3 uniform with boils and irregularities
4 pool and riffle sequence
5 tumbling flow

Bar type:
0 0 none
1 1 point bars
2 2 side bars
3 3 mid-channel bars
4 4 diagonal bars
5 5 large dunes

Meander dimensions:
belt width ______ mi.
wave length ______ mi.
sinuosity ______

Natural obstructions:
0 0 none
1 1 logs
2 2 beaver dams
3 3 boulders (lag material)
4 4 vegetation

Degree of obstruction:
0 0 none
1 1 occ. minor
2 2 occ. major
3 3 frequent minor
4 4 frequent major

Comments: Small dam

Lateral Channel Activity

Lateral activity:
0 not detectable
1 downstream progression
2 progression and cut-offs
3 mainly cut-offs
4 entrenched loop development
5 laterally active but not 1-4

Lateral stability:
0 stable
1 slightly unstable
2 moderately unstable
3 highly unstable

Comments: ______

Channel Banks and Bed

Alluvial bank material:
0 0 0 no alluvial banks
1 1 1 clay and silt (cohesive)
2 2 2 silt and sand (non-cohesive)
3 3 3 sand and gravel (< 64 mm)
4 4 4 sand to cobbles
5 5 5 sand overlain by silt
6 6 6 gravel overlain by silt
7 7 7 cobbles overlain by silt

Non-alluvial bank material:
0 0 0 alluvial bank material
1 1 1 lacustrine deposits
2 2 2 till
3 3 3 easily erodible rock
4 4 4 moderately erodible rock
5 5 5 resistant rock
6 6 6 boulders

Percentage of left bank in alluvium ______ %
Percentage of right bank in alluvium ______ %

Bank vegetation:
0 none
1 weak
2 good
3 very strong

Predominant bed material:
1 sand
2 sand with local gravel
3 gravel
4 gravel with local sand
5 sand and gravel

Depth of alluvium:
0 no alluvium
1 shallow
2 moderate
3 deep

Estimated depth of alluvium ______ ft.

Reference or comments: ______

Bed Rock Below Channel

Presence of rock outcrops in channel bed:
0 none
1 one occurrence
2 two occurrences
______ occurrences
9 several occurrences

Rock type at channel base:
0 0 0 not applicable (none for great depth)
1 1 1 compact clay
2 2 2 shale
3 3 3 limestone
4 4 4 sandstone
5 5 5 conglomerate
6 6 6 granite
7 7 7 ______

Erodibility:
0 0 0 not applicable
1 1 1 soft cohesive
2 2 2 easily erodible
3 3 3 moderately erodible
4 4 4 resistant

Comments: ______

Figure 9.69c: Long Profile of Grand River (Ontario).

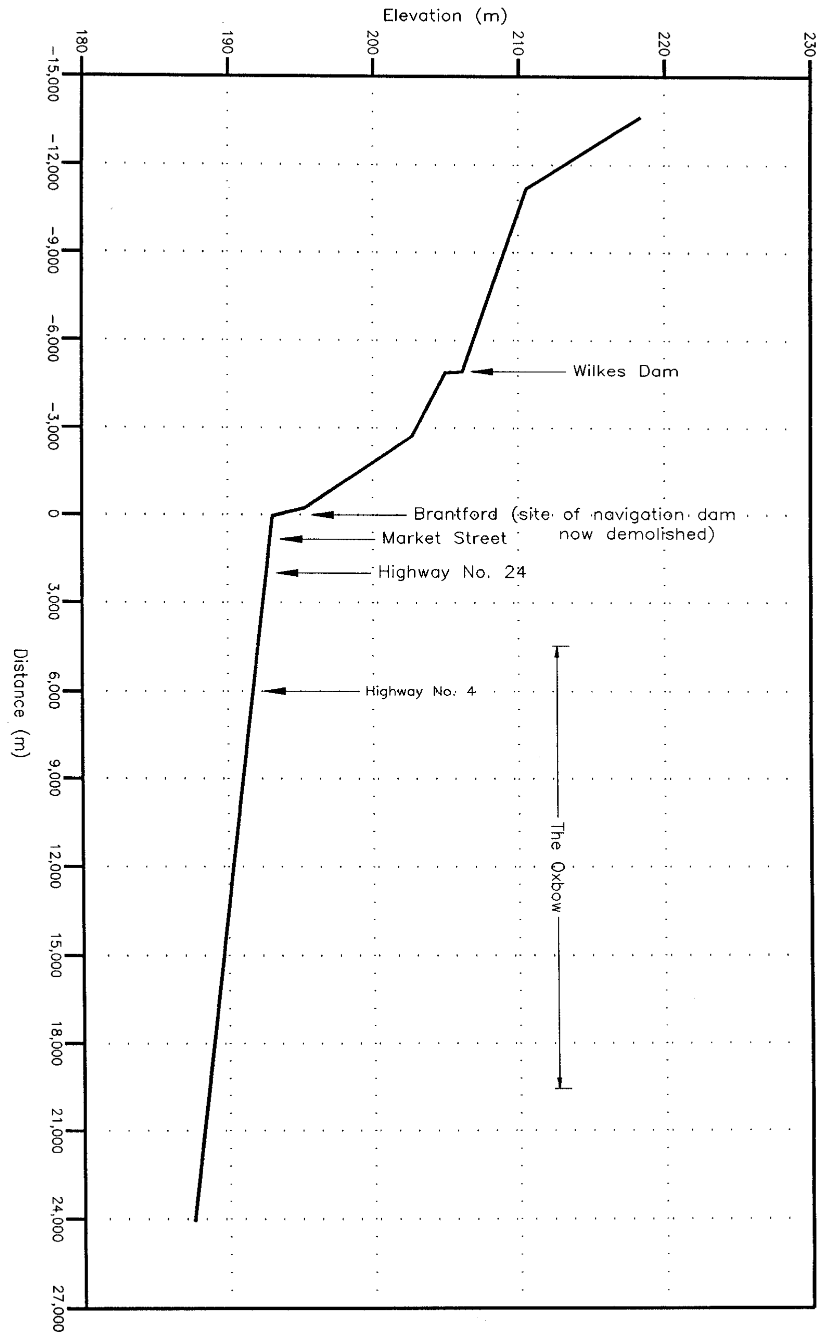

Figure 9.70: Grand River, Ontario.

Bridge crossing at hwy #4 - Erie Avenue south of Brantford. The bridge has one pier in the middle of the channel and a high deck with open spans on the floodplain. Flow is right to left. Note slump of terrace right bank downstream from bridge.

Figure 9.71: Contour Map of River.

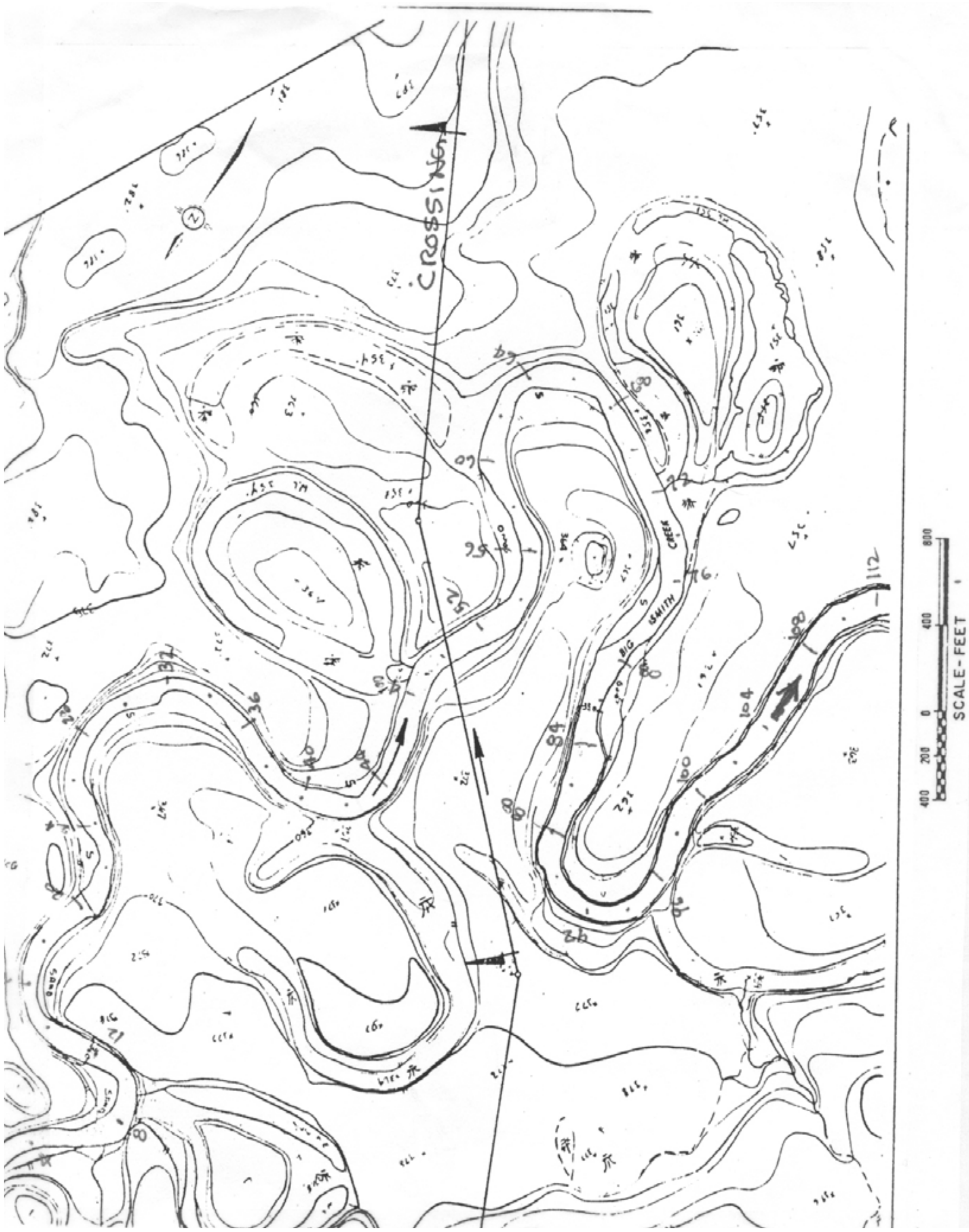

Figure 9.72: Comparison of Air Photos.

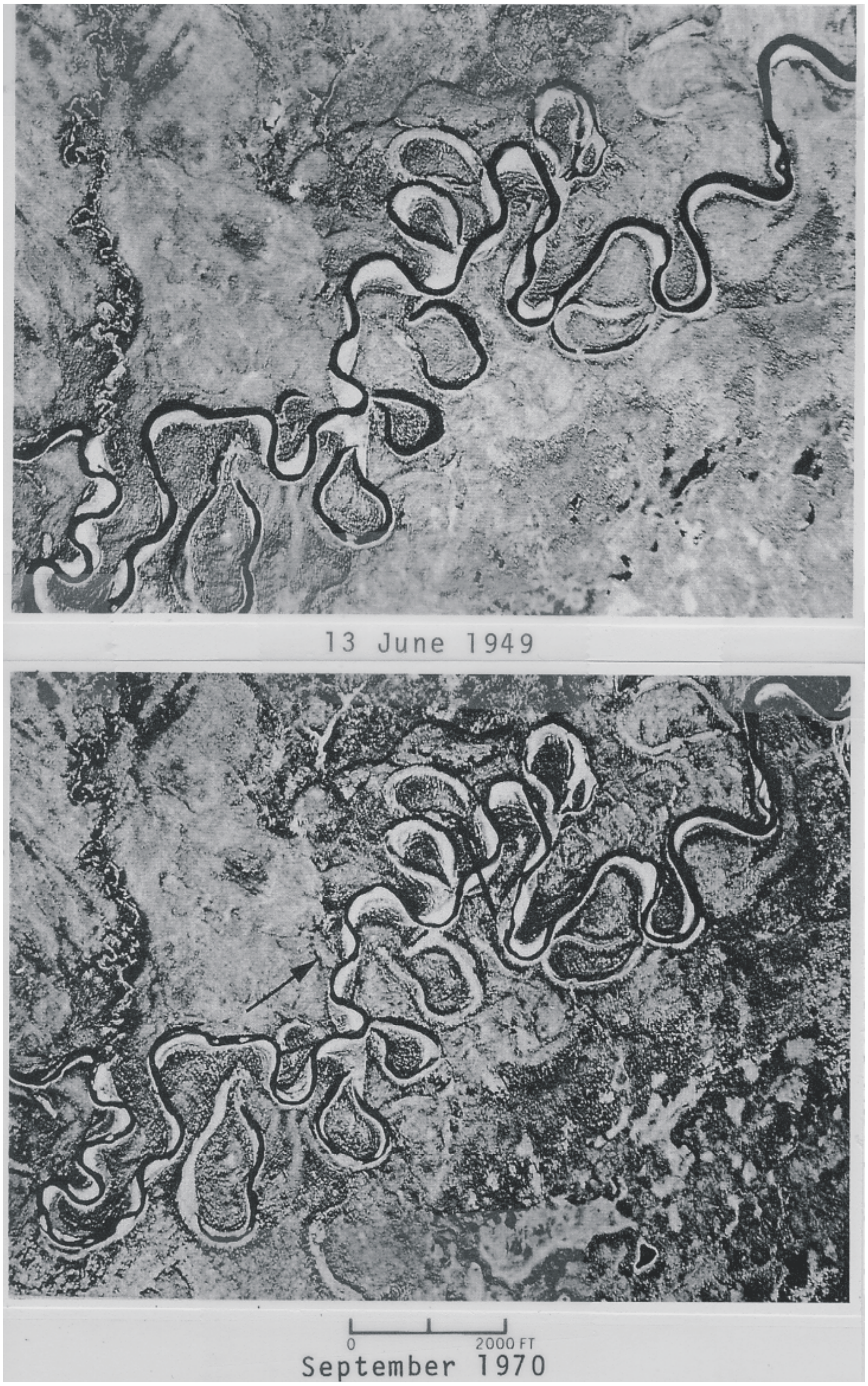

Figure 9.73: River in Northern Canada.

View of tortuous meander pattern just downstream from proposed bridge crossing. Note that at A there exists a bedrock waterfall which is shown on the next photo (Fig. 9.74)

Flow is left to right.

Figure 9.74: River in Northern Canada.

View of bedrock waterfall downstream from proposed bridge crossing. The waterfall is about 10 feet high. Note the loop-cut will take place at B which will result in a sudden headcut working upstream.

Flow is right to left.

Figure 9.75: River in Northern Canada.

Development of gravel-sand point bar at inside of bend.

Flow is left to right.

Figure 9.76: River in Northern Canada.

Bed-material, sand and gravel on point bar.

Figure 9.77: Long Profile of Northern River (Canada).

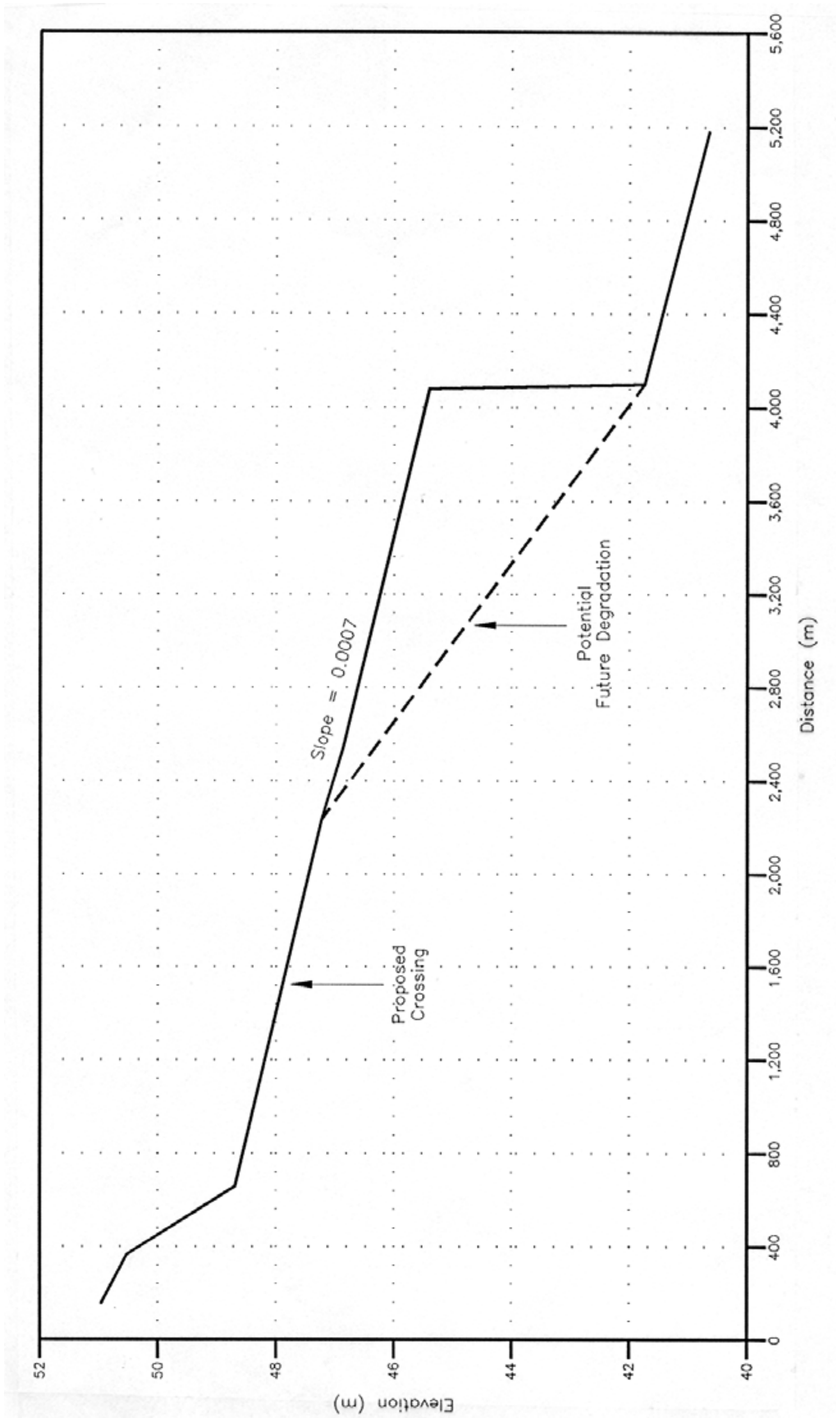

Figure 9.78a: River Data Sheet No. Geog1

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DEPARTMENT OF CIVIL ENGINEERING

RIVER DATA SHEET No. Geog. 1/71

GEOGRAPHIC FEATURES

Reach Name: NORTHERN RIVER IN CANADA Reach No:________ Date of Analysis:______________ Analysis By:__________________

Scale of Air Photos:____________________ Scale of Map:____________________

NOTE: Complete codes by circling the appropriate number(s). Use "-1" for "unknown" and "0" for "not applicable".

General Description of the Terrain in the Vicinity of the Surveyed Reach, above Valley

Terrain:
1 mountainous
2 foothills
3 uplands
4 hills
5 plains
6 lowlands

Vegetation:
0 0 0 not applicable
1 1 1 almost none
2 2 2 grass
3 3 3 shrubs
4 4 4 sparsely forested, 0-25%
5 5 5 moderately forested, 25-75%
6 6 6 heavily forested, 75-100%
7 7 7 swamp or muskeg

Forest type:
0 0 0 not applicable
1 1 1 deciduous
2 2 2 coniferous

Comments:____________________

Land use:
0 0 no cultivation or built-up area
1 1 partly cultivated
2 2 mainly cultivated
3 3 partly built-up
4 4 urbanized

Surficial geology:
1 1 1 bedrock
2 2 2 ground moraine
3 3 3 hummocky moraine
4 4 4 lacustrine deposits
5 5 5 glacio-fluvial dep.
6 6 6 fluvial deposits
7 7 7 aeolian deposits

Valley Characteristics above Valley Flat

Valley measurements:
____within reach
____within reach and immediate vicinity
depth:__________ft.
top width:__________mi.
bottom width:__________mi.

Slumping of valley walls:
0 none
1 occasional
2 frequent

Length of reach with slumping valley walls (contact length in percent of total length of banks):__________

Vegetation on valley wall:
0 0 not applicable
1 1 almost none
2 2 grass
3 3 shrubs
4 4 sparsely forested
5 5 moderately forested
6 6 heavily forested
7 7 swamp or muskeg

Forest type on valley wall:
0 0 not applicable
1 1 deciduous
2 2 coniferous

Comments:____________________

Terraces

Terrace presence:
0 none
1 indefinite
2 fragmentary
3 continuous

Number of levels:
0 not applicable
1 one level
2 two levels
9 several levels
_ ___ levels

Comments (in particular land use and vegetation):____________________

Relation of Channel to Valley

Valley type:
0 not applicable
1 stream cut valley
2 stream cut valley in wide valley
3 wide mountainous valley

If no valley:
0 valley present
1 on alluvial fan
2 on alluvial plain
3 in delta
4 in old lake

Underfit:
0 not applicable or not obviously underfit
1 obviously underfit

Local lateral constriction:
0 none
1 one
2 two
____ cases
9 several cases

Relation of channel to valley bottom (vertical):
0 not applicable
1 not obviously degrading or aggrading
2 partly entrenched
3 entrenched
4 aggrading

Relation of channel to valley walls or to high, resistant terraces (lateral):
0 not applicable (no valley or free)
1 occasionally confined
2 frequently confined
3 confined
4 entrenched

Comments:____________________

Figure 9.78b: River Data Sheet No. Geog2

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DEPARTMENT OF CIVIL ENGINEERING

RIVER DATA SHEET No. Geog. 2/71

GEOGRAPHIC FEATURES - (Cont'd.)

Reach Name: NORTHERN RIVER IN CANADA Reach No: ______

Description of Valley Flat

Presence:
0 none
1 indefinite
2 fragmentary
3 continuous

Extent:
0 none
1 narrow (< 1 W_s)
2 moderate (1-5 W_s)
3 wide (> 5 W_s)

Average width ______ mi.
Maximum width ______ mi.
Channel length with valley flat on left ______ %
on right ______ %

Vegetation:
0 0 not applicable
1 1 almost none or bare
2 2 grass
3 3 shrubs
4 4 sparsely forested
5 5 moderately forested
6 6 heavily forested
7 7 swamp or muskeg

Forest type:
0 0 not applicable
1 1 deciduous
2 2 coniferous

Land use:
0 0 not cultivated, not built-up
1 1 partly cultivated
2 2 mainly cultivated
3 3 partly built-up
4 4 mainly built-up

Comments: ______

Channel Description (near long-term mean)

Channel pattern:
1 straight
2 sinuous
3 irregular
4 regular meanders
5 irregular meanders
6 tortuous meanders

Islands:
0 none
1 occasional
2 frequent
3 split
4 braided

Type of flow:
1 uniform water surface
2 uniform with rapid in reach
3 uniform with boils and irregularities
4 pool and riffle sequence
5 tumbling flow

Bar type:
0 0 0 none
1 1 1 point bars
2 2 2 side bars
3 3 3 mid-channel bars
4 4 4 diagonal bars
5 5 5 large dunes

Meander dimensions:
belt width ______ mi.
wave length ______ mi.
sinuosity ______

Natural obstructions:
0 0 none
1 1 logs
2 2 beaver dams
3 3 boulders (lag material)
4 4 vegetation

Degree of obstruction:
0 0 none
1 1 occ. minor
2 2 occ. major.
3 3 frequent minor
4 4 frequent major

Comments: ______

Lateral Channel Activity

Lateral activity:
0 not detectable
1 downstream progression
2 progression and cut-offs
3 mainly cut-offs
4 entrenched loop development
5 laterally active but not 1-4

Lateral stability:
0 stable
1 slightly unstable
2 moderately unstable
3 highly unstable

Comments: SLOW PROCESS

Channel Banks and Bed

Alluvial bank material:
0 0 0 no alluvial banks
1 1 1 clay and silt (cohesive)
2 2 2 silt and sand (non-cohesive)
3 3 3 sand and gravel (< 64 mm)
4 4 4 sand to cobbles
5 5 5 sand overlain by silt
6 6 6 gravel overlain by silt
7 7 7 cobbles overlain by silt

Non-alluvial bank material:
0 0 0 alluvial bank material
1 1 1 lacustrine deposits
2 2 2 till
3 3 3 easily erodible rock
4 4 4 moderately erodible rock
5 5 5 resistant rock
6 6 6 boulders

Percentage of left bank in alluvium ______ %
Percentage of right bank in alluvium ______ %

Bank vegetation:
0 none
1 weak
2 good
3 very strong

Predominant bed material:
1 sand
2 sand with local gravel
3 gravel
4 gravel with local sand
5 sand and gravel

Depth of alluvium:
0 no alluvium
1 shallow
2 moderate
3 deep

Estimated depth of alluvium ______ ft.

Reference or comments: ______

Bed Rock Below Channel

Presence of rock out-crops in channel bed:
0 none
1 one occurrence
2 two occurrences
______ occurrences
9 several occurrences

Rock type at channel base:
0 0 0 not applicable (none for great depth)
1 1 1 compact clay
2 2 2 shale
3 3 3 limestone
4 4 4 sandstone
5 5 5 conglomerate
6 6 6 granite
7 7 7 ______

Erodibility:
0 0 0 not applicable
1 1 1 soft cohesive
2 2 2 easily erodible
3 3 3 moderately erodible
4 4 4 resistant

Comments: ______

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Ministry of Transportation
Ontario

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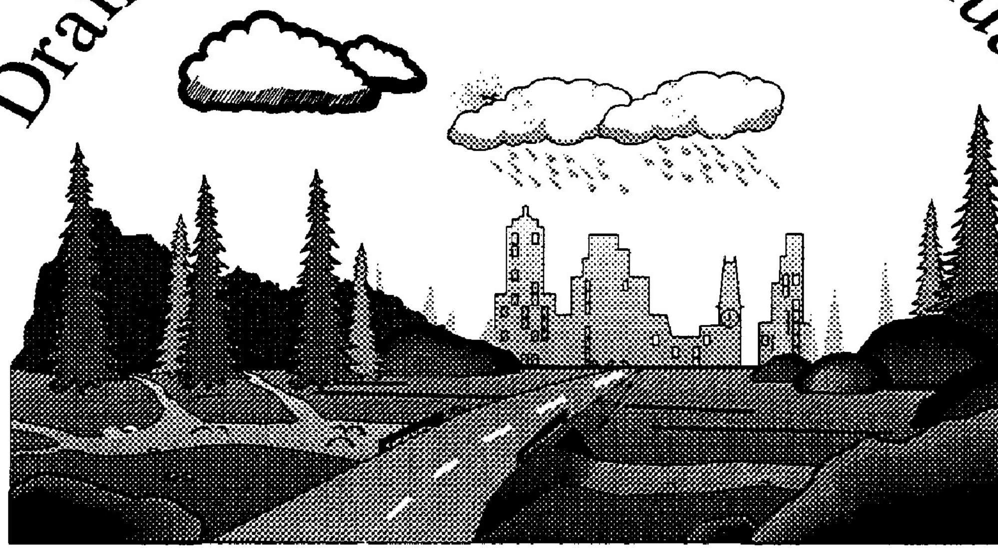

Chapter 10
Introduction to Soil Bioengineering

Drainage and Hydrology Section
Transportation Engineering Branch
Quality and Standards Division

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Introduction

In terms of engineering practice, soil bioengineering could be described as a form of *Adaptive Environmental Management*. It is an activity that is largely untried on roadside applications throughout much of the province. As such, projects should be designed and implemented using the best available information, then monitored, evaluated, and rebuilt or modified as required to achieve the desired objectives. The advancement of soil bioengineering as a valid method of establishing vegetative cover and minimizing erosion is dependent on careful selection of trial sites, implementing projects, and evaluating the results. Any projects carried out should be well documented, so they can become part of the growing knowledge base on the application of soil bioengineering in Ontario conditions.

Purpose of the Chapter

The purpose of this chapter is to provide practitioners with preliminary information for considering whether the emerging technology of soil bioengineering can be an alternative or augmentation to traditional erosion control works to protect stream banks in highway projects. As soil bioengineering should be used only on slopes which have overall mass stability, it is not an intention of this chapter to encourage replacement of earth retaining structures with soil bioengineering solutions.

Consistent with its purpose, this chapter focuses on outlining some of the design considerations required to successfully select and implement a soil bioengineering system, rather than presenting a step by step design approach. The requirement for a multidisciplinary team to implement a soil bioengineering project is emphasized throughout the chapter. The team is required not only to design and implement the project, but to monitor and evaluate its success. The causes of erosion are examined and potential solutions are proposed. Although some of the appropriate plant species are identified, knowledge of local growing conditions is critical to successful project implementation.

The Benefits of Soil Bioengineering

The use of live plant material provides an aesthetically pleasing, natural method of erosion control. Soil bioengineering systems are designed to strengthen over time, as root mass builds up and binds the soil. Live plant materials help make the systems self-repairing in the event of minor washouts. Woody vegetation acts to loosen the soil and create macropores, allowing for greater infiltration. This results in a reduction in surface runoff and the erosion it causes.

The application of soil bioengineering may enhance vegetation to support wildlife. When used to reduce erosion on a stream bank, soil bioengineering systems provide shade to the stream. This may result in a reduction in the average water temperature, allowing a wider diversity of species in the stream. Soil bioengineering is a natural tool that can be used in conjunction with the natural channel design techniques outlined elsewhere in this manual.

At this point it is difficult to quantify, but there are reports that soil bioengineering is less expensive to install than some of the more traditional erosion control techniques. This statement should be viewed with caution, since there are few Ontario examples to draw on. Accurate determination of the cost/benefit of soil bioengineering will require the implementation and assessment of a number of projects.

The Limitations of Soil Bioengineering

Although there are examples of successful soil bioengineering projects throughout the United States and Europe, the techniques have not been widely applied in Ontario. Because of this, there have been no formal design approaches developed for Ontario. It is important that the practitioner recognizes that the techniques cannot normally be used to steepen a slope beyond its natural angle. Although the woody vegetation may have the effect of drying the slope material and contributing to the overall mass stability, it is recommended that the influence of the woody vegetation be assumed to be negligible when soil stability calculations are carried out. Caution should be used when selecting plant species for roadside application. Species that attract wildlife to the roadside may cause undesirable traffic hazards.

Project timing requirements pose further limitations on the installation of soil bioengineering projects. Construction must be carried out during the plants' dormant period, which extends from approximately late September until mid-April in Ontario. Frozen ground conditions during much of this period further limit the construction window to a few weeks in the fall, and a few weeks in the spring each year. Depending on the location of the project, this period may be inconsistent with fisheries guidelines.

Chow (1959) lists Manning's n values for various channel configurations and linings. In general, banks with dense vegetation cover tend to have higher n values than concrete lined, or riprap lined channels. This is of particular concern on smaller channels. The practitioner must consider the impact of increased channel roughness when designing any conveyance channel.

A final limitation results from the fact that soil bioengineering requires the use of live plant materials. Few contractors or public works crews have experience in dealing with the large quantities of woody vegetation cuttings required on a soil bioengineering project. As such, the success of any soil bioengineering installation is highly dependent on close site supervision by an experienced practitioner.

Application of Soil Bioengineering

This section of the chapter focuses on identifying the cause of erosion and selecting an appropriate solution. The selection and implementation of a soil bioengineering solution depends on a thorough understanding of the cause of erosion and the site conditions. Refer to Chapters 5 and 8 for a more detailed discussion of erosion.

Identifying the Cause of Erosion on Roadside Embankments

The Universal Soil Loss Equation (USLE) provides a guideline for the loss of soil due to rainfall and runoff in a sloped area (Schwab et al). Although use of the equation may be inappropriate for calculating soil loss from an individual site, it does provide a good overview of the factors that contribute to soil loss. The equation states that soil loss is a function of R,K,LS,C, and P, where

R	= Rainfall and runoff erosivity index based on site location.
K	= Soil erodibility factor based on the soil type.
LS	= Topographic factor based on the slope length and gradient.
C	= Cropping factor that relates to the cover on the soil (also called the V-M factor).
P	= Conservation practice factor related to the direction of tillage.

Table 10.1 provides some general information on how an examination of each of the factors in the USLE can provide a guideline for minimizing surficial erosion on an embankment. The overall assumption is that the slope has been graded to a stable angle.

It is important to recognize the value of the USLE in developing concepts for reducing erosion on roadside embankments. The C factor can be thought of as vegetative management practices that prevent or reduce soil detachment and erosion. These practices include non-structural measures, such as the establishment of vegetative cover. The P factor can be thought of as practices that prevent or reduce the movement of soil particles once they have been detached. These practices include the establishment of vegetation using soil bioengineering techniques, as well as the structural methods outlined earlier in this manual. The reader is referred to Chapter 8 for a more detailed discussion of the Universal Soil Loss Equation.

In addition to surficial erosion caused by water flow, unvegetated roadside embankments can be subject to wind erosion. Vegetative cover on the slope acts to reduce wind velocities; root material acts to bind and confine the soil layer. Other erosive damage (piping) can occur as a result of water emerging from the face of the slope, carrying with it fine soil particles. Soil bioengineering methods can be applied to reduce erosion caused by emerging water. In addition to binding the soil, properly selected plants can have a drying effect on the bank material.

Table 10.1: USLE Factors

USLE Factor	Cause of Erosion	Options for Minimizing Erosion
R	• Rainfall impact and runoff cause detachment and down-slope transport.	• R based on geographic factors.
K	• Soil strength insufficient to withstand erosive agent.	• Mechanical compaction of surface layer. • Woody vegetation roots to bind and restrain soil.
LS	• Long slope length. • Steep slope angle.	• Reduce slope length by terracing or benching. • Grade slope to less steep angle.
C	• Bare slope subject to rainfall and runoff erosion.	• Establish cover to intercept rainfall and reduce velocity of down-slope runoff.
P	• Rills form in machine tracks.	• Carry out finish grading normal to slope angle.

Some of the applications of soil bioengineering to stabilize an embankment include:

- live cribwalls to construct terraces;
- live fascines and brush layers to break up the slope and reduce runoff velocity;
- live staking to promote early colonization by woody vegetation; and
- brush layering to anchor the surface layer of the soil.

There are other applications of soil bioengineering, for example, coir logs, but it is outside the scope of this manual to mention them all.

Identifying the Cause of Stream Erosion

A stream in equilibrium could be described as a stream that has sufficient energy to carry the water and sediment loads imposed by its watershed. Even streams in equilibrium experience bank erosion. Changes in land use that increase the frequency or duration of flooding tend to accelerate erosion. In highly erodible soils, minor disturbance to the vegetative cover can lead to serious erosion.

Identifying the cause of stream erosion is the first step in developing a solution. Table 10.2 provides information on the various causes of stream erosion. Soil bioengineering techniques can be effective for dealing with many of these causes of erosion. Chapter 5 outlines a number of **traditional** erosion control techniques. Soil bioengineering can be used to augment these solutions. Also, the live plant material will provide other benefits like stream shading and wildlife habitat. In effect, soil bioengineering can be used to **soften** a hard erosion control structure.

Table 10.2: Causes of Stream Erosion

Type of Erosion	Cause
• Toe erosion and upper bank failure.	• Removal of non-cohesive toe materials followed by over steepening and failure of upper slope.
• Bed scour over extended reaches.	• Increased flow due to upstream land use changes. • Changes in stream gradient due to relocation. Tends to occur at mouth of stream when receiving stream is dredged or lowered.
• Head cutting.	• Increased flow due to upstream land use changes. • Decreased resistance due to reduced bank cover.
• Mid bank and upper bank scour.	• Local obstruction. • Loss of vegetation.
• Local bank scour.	• Local obstruction, often found below culvert outfalls.
• Local bed scour.	• Poorly compacted soil around culverts/pipes. • Quick drawdown leading to steep hydraulic gradient in the vicinity of the bank.
• Piping.	• Insufficient protection in a zone of concentrated flow. Source: Nunnally (1990).

Soil Erodibility

Soil erodibility can be described in terms of a soil's ability to resist the shear forces that cause erosion. Factors affecting erodibility are the particle size, interparticle cohesion, moisture content, organic matter content, and plasticity. These factors are discussed in greater detail in Chapter 8. The Ontario Ministry of Agriculture, Food and Rural Affairs has listed the relative susceptibility to water erosion for a number of soil textures commonly found in Ontario. In general, non-cohesive fine texture soils tend to be highly erodible, cohesive soils tend to have medium erodibility, and soils with coarse texture tend to be the least erodible. Table 10.3 provides a guide to the relative erodibility of the various soil textures that would be encountered on projects in Ontario.

Table 10.3: Relative Erodibility Guide

Surface Soil Texture		Susceptibility to Erosion
Very fine sand.	**More erodible**	Very high.
Loamy very fine sand. Silt loam. Very fine sandy loam. Silty clay loam.		High.
Clay loam. Loam. Silty clay. Clay. Sandy clay loam. Fine sand. Heavy clay.		Medium.
Sandy loam. Loamy fine sand. Fine sand. Coarse sandy loam.		Low.
Loamy sand. Sand.	**Less erodible**	Very low.

The Role of Vegetation in the Protection of Slopes

Erosion and sedimentation can be viewed as a three part process involving: detachment of the soil particles, transport of the eroded material, and deposition of material when there is insufficient energy to continue transport. One mechanism that results in soil particle detachment is high energy rainfall impacting on the surface and splashing soil particles down-slope. The other major detachment mechanism occurs when overland runoff has sufficient velocity to generate shear stresses in excess of the soil's shear strength. Both processes have the net effect of transporting soil down the slope. Vegetation can minimize detachment and transport. Live vegetation reduces detachment by absorbing rainfall energy before it hits the surface, while surface debris or residue acts to slow runoff, reducing the shear stresses and the resulting detachment. The reduced velocity also allows some of the eroded material in transport to settle out. In addition to intercepting rainfall, woody vegetation provides mechanical reinforcement for an embankment or stream bank, reducing the erosion potential. Another impact of vegetation is its drying effect on the soil, allowing the soil to more readily infiltrate rainfall. The net effect is a delay in the onset of runoff. Tables 10.4 to 10.6, modified from information found in Gray and Leiser (1982), show impacts of vegetation.

Table 10.4: Major Impacts of Herbaceous Vegetation

Process	Impact on Erosion
• Interception.	• Plants absorb rainfall energy. • Plant roots and debris prevent soil compaction, • allowing an increase in infiltration.
• Restraint.	• Root systems bind soil particles. • Residue filters sediment out of runoff.
• Retardation.	• Residue increases surface roughness. • Reduces velocity of runoff.
• Infiltration.	• Roots and residue help maintain soil porosity. • Delay the onset of runoff.
• Evapotranspiration.	• Depletion of soil moisture. • Delay the onset of runoff.

Table 10.5: Possible Negative Impacts of Woody Vegetation

Process	Impact on Erosion
• Surcharge.	• Can exert destabilizing down-slope stresses.
• Root wedging.	• Roots invading cracks and fissures can reduce local stability due to prying/wedging action.
• Windthrowing.	• Destabilizing influence due to large turning moments exerted by strong winds on trees.

Selecting a Soil Bioengineering Solution

There are a number of factors that must be considered prior to designing and implementing a soil bioengineering project. Input from a wide variety of professional disciplines is required. Typical project teams would include a geotechnical engineer, a soils/vegetation specialist, a landscape architect, a biologist, a hydrologist, and in some cases a geomorphologist. In addition, project construction should be supervised by someone accustomed to working with live cuttings. Table 10.7 outlines the services of some of the key project team members.

Table 10.6: Positive Impacts of Woody Vegetation

Process	Impact on Erosion
• Root reinforcement.	• Mechanical reinforcement. • Transfer of shear stress in the soil to tensile stress in the plant root.
• Soil moisture modification.	• Evapotranspiration and interception reduce buildup of soil moisture stress. • May cause decreased rate of snowmelt.
• Buttressing and arching.	• Anchored roots may act as arch abutments. • Counteracts shear stresses.
• Surcharge.	• Surcharge can increase resistance to sliding.

Table 10.7: Services of Some of Key Members of Project Team

Team Member	Services
• Geotechnical engineer.	• Evaluate soil conditions on site. • Assess the overall stability of the roadside embankment or stream bank.
• Hydrologist.	• Assess flow conditions and determine design flow. • Evaluate the impact of vegetation on flow capacity.
• Landscape architect, vegetation specialist and biologist.	• Select appropriate species for project. • Locate sources of live plant materials. • Evaluate soil fertility on the site. • Evaluate potential of proposed solution to enhance aquatic or terrestrial habitats.
• Geomorphologist.	• Evaluate conditions of stream and sedimentation.
• Construction superintendent.	• Organize and oversee project construction. Must be familiar with handling, transportation and installing live cuttings.

In addition to assembling a diverse team of specialists, the successful implementation of a soil bioengineering project requires careful consideration of other factors listed in Table 10.8.

Table 10.8: Guide to Design Considerations

Factor	Possible Considerations
• Mass stability.	• Slope must be graded to a stable angle prior to implementing the project. • Soil bioengineering is not normally used to steepen a slope beyond the soil's natural stable angle.
• Soil texture.	• Soil must have texture suitable for compaction around the plant material. • It may be necessary to dampen soil to achieve adequate compaction.
• Soil orientation.	• Exposure to sunlight required. • Cuttings do not thrive in highly shaded areas.
• Surface drainage.	• Areas of concentrated flow must be identified and treated to resist erosive forces.
• Plant species.	• Local supply of woody vegetation required. • Plants must root easily from cuttings. • Rooted stock may be used to enhance a soil bioengineering project.
• Project timing.	• Projects must be carried out in the dormant period when the ground is not frozen, late fall or early spring. • For stream stabilization projects, timing may conflict with fisheries guidelines.
• Soil moisture.	• Plant species must be consistent with the moisture conditions on the slope.

Soil Bioengineering Solutions

Three main applications for which soil bioengineering may be appropriate are listed in Table 10.9. For each of the three applications identified, there are a number of potential soil bioengineering solution techniques. Six potential techniques are presented in this section and Table 10.10 provides a guideline for selecting appropriate techniques for various applications.

Table 10.9: Main Applications of Soil Bioengineering

Application	Project Goal	Typical Project
• Embankment stabilization.	• Protect slope face from surficial movement.	• Horizontal highway realignment. • Vertical highway realignment. • Interchange construction.
• Stream bank protection.	• Minimize erosion on the banks of stream.	• Culvert or bridge inlet and exit areas. • Newly realigned streams.
• Surficial erosion protection.	• Minimize erosion from surface waters.	• Surface drainage.

Table 10.10: Guide to Selection of Solution

Bioengineering Technique	Embankment Stabilization	Stream bank Protection	Surface Drainage
• Live fascines.		✓	
• Brush layer.	✓		✓
• Live staking.	✓	✓	
• Riprap with brush layer.		✓	✓
• Riprap with live staking.		✓	
• Live cribwall.	Can be used to build terrace.	✓	

Although Table 10.10 provides a guide, the judgment of an experienced practitioner is required to select a solution appropriate for a specific site. The following describes the six techniques. It is useful to note that combining soil bioengineering with more traditional erosion control measures is a low risk method for a practitioner to become familiar with soil bioengineering.

Live Fascine

Description A live fascine is a long bundle of branch cuttings, tied together with twine. The bundle has 150 to 200 mm diameter and is typically placed in a shallow trench running along the slope. The trench is backfilled and soil must be well compacted around the fascine. The majority of the fascine is normally buried. Fascine spacing is based on slope length and steepness.

Applications Live fascines are used on slopes with the goal of establishing quick cover and minimizing surficial movement. Live fascines can also be used to stabilize the edge of a stream bank. See Figures 10.1 and 10.2. It is expected that a portion of the cuttings will form roots and grow. Using rooted stock for a portion of the plant material in the fascine can increase the number of plants that become established. Eventually, the fascine forms a line of well-rooted woody vegetation. Chapter 18 of the USDA *Engineering Field Handbook* (1992) contains guidelines (see Table 10.11) for live fascine spacing. The dimensions have been converted to SI units.

Table 10.11: USDA Live Fascine Spacing Guidelines

Slope (H:V)	Distance between Trenches (m)	Max. Slope Length (m)
1:1 to 1.5:1	0.9 - 1.2	4.6
1.5:1 to 2:1	1.2 - 1.5	6.1
2:1 to 2.5:1	1.5 - 1.8	9.1
2.5:1 to 3:1	1.8 - 2.4	12.2
3.5:1 to 4:1	2.4 - 2.7	15.2
4.5:1 to 5:1	2.7 - 3.0	18.3

Installation Guidelines The live fascine is constructed adjacent to a trench dug along the contour of the slope. Trenches are normally 150 to 200 mm deep, and are often dug by hand. The fascine is laid in the trench and anchored in place with live stakes. It is critical that the soil is well compacted around the fascine to maximize contact between the plant cutting and the soil. Figure 10.3 illustrates a live fascine, along with the installation configuration for live fascines on a slope.

Figure 10.1: Live Fascine Used to Stabilize Edge of Small Stream

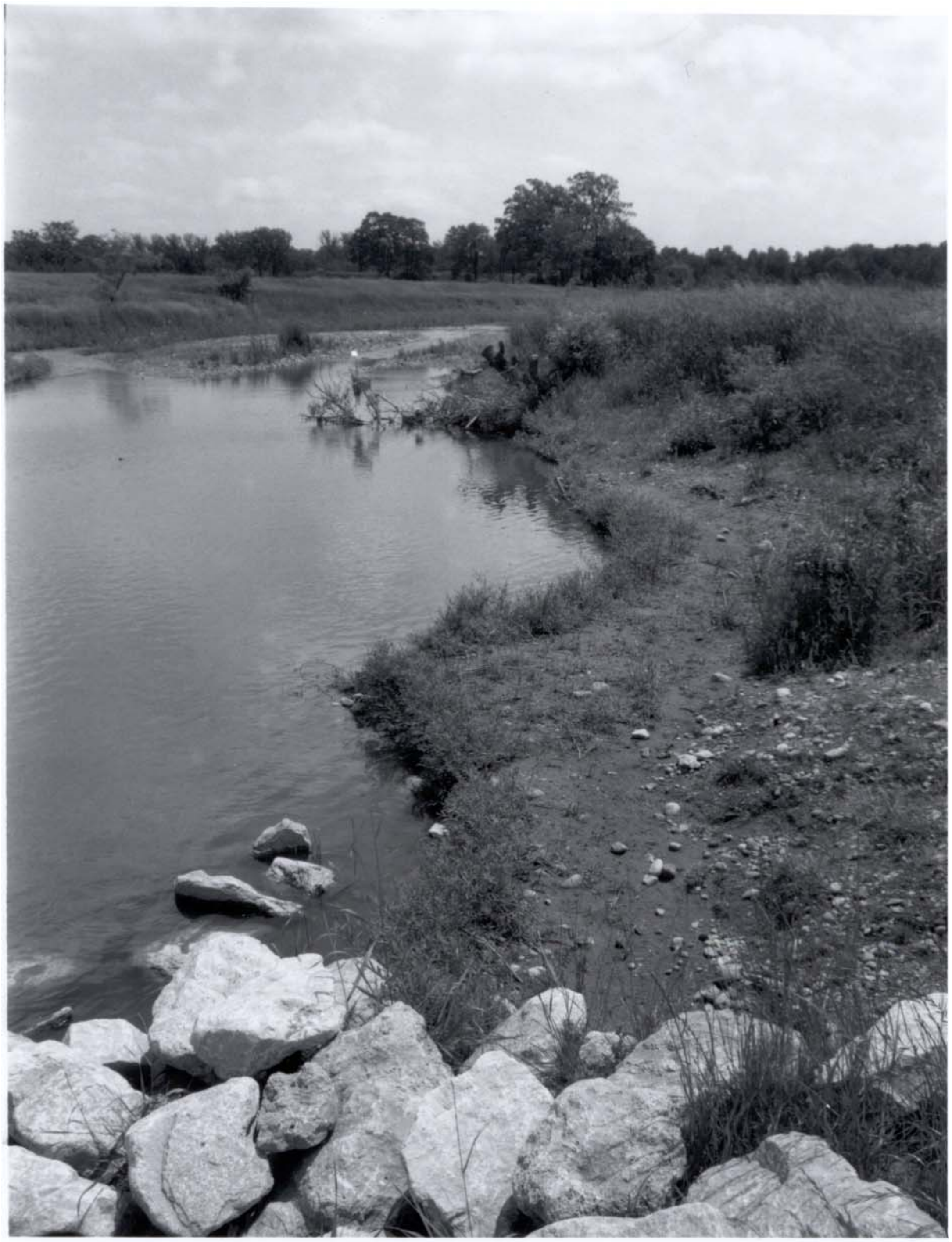
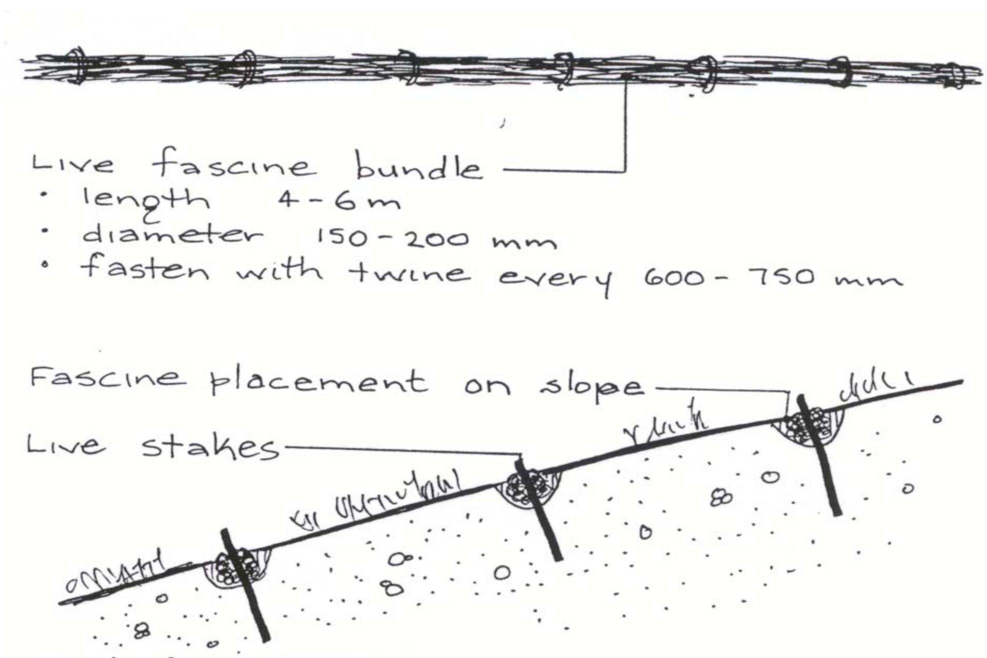


Figure 10.2: Live Fascine on Contour after First Year Growth

Figure 10.3: Schematic Diagram of a Live Fascine Installation



Brush Layer

Description The brush layer consists of a layer of brush that angles back into the slope, perpendicular to the contour. Typical installations extend 0.9 to 1.8 m back into the slope.

Applications Brush layers are typically used to break up a slope into a series of shorter slopes separated by layers of brush. The technique can be used on both cut slopes and fill slopes. See Figures 10.4 and 10.5. A brush layer may be combined with a fascine to provide protection along the edge of the stream. Table 10.12, modified from Chapter 18 USDA *Engineering Field Handbook* (1992), provides guidelines for spacing brush layers on a slope.

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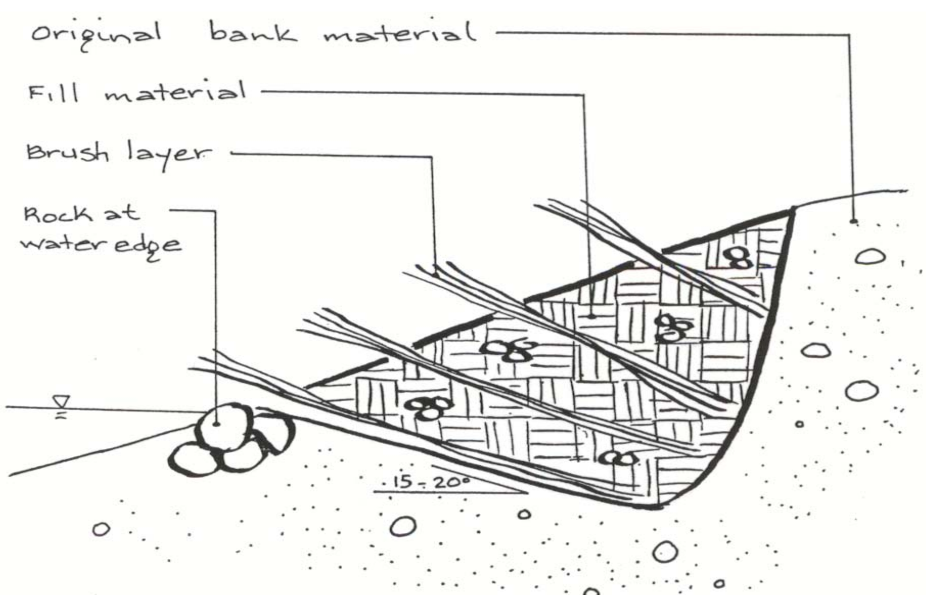
Table 10.12: Brush Layer Spacing Guidelines

Slope (H:V)	Slope Distance between Benches		Max. Slope Length (m)
	Wet Slope (m)	Dry Slope (m)	
2:1 to 2.5:1	0.9	0.9	4.6
2.5:1 to 3:1	0.9	1.2	4.6
3.5:1 to 4:1	1.2	1.5	6.1

Figure 10.4: Schematic Diagram of a Brush Layer Installation



Installation Guidelines Brush layer installation normally begins at the toe of the slope. A small bench is formed along the contour of the slope. The bench is typically angled 15 ^{o} to 20^{o} to horizontal with the outside edge being higher than the inside edge. Brush is laid on the benched area in a criss-crossed pattern, with the growing tips generally oriented away from the slope. Soil is then compacted in and around the brush to maximize contact between the soil and the plant material. Often, soil excavated from the next bench up the slope is used to cover the brush. After backfilling is completed, the brush is cut off so that only 100 to 150 mm extends from the slope.

For slopes steeper than 3:1 slope, it is recommended that jute mesh is used to wrap each soil layer (USDA *Engineering Field Handbook*, 1992).

Figure 10.5: Brush Layer Used to Stabilize a Spillover Area

Live Staking

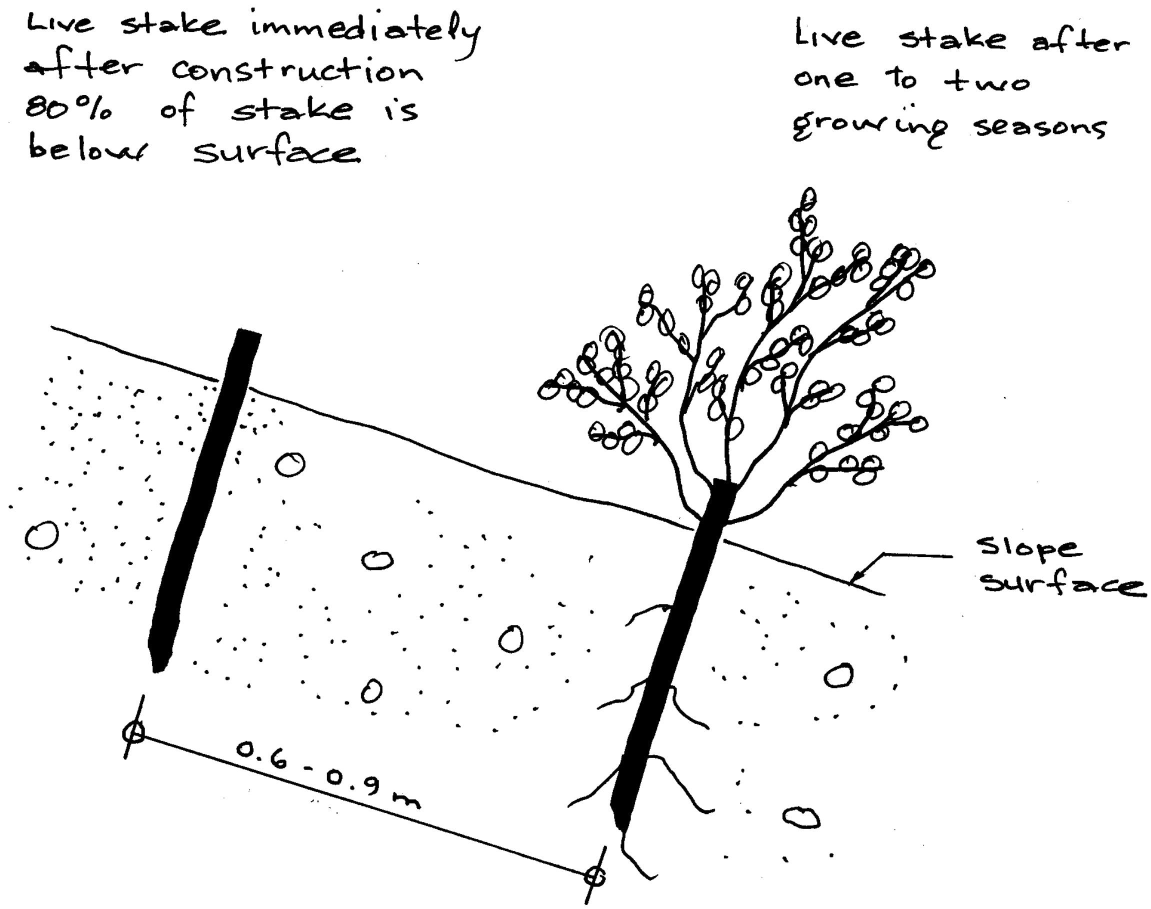
Description A live stake is 25 to 50 mm in diameter and 600 to 900 mm long. The stake is cut from a woody vegetation species that roots easily from cuttings. The basal end of the stake is normally cut off at a 45 ^{o} angle. The upper end is cut off square. It is important that the stake is not damaged during installation.

Applications Live stakes are typically used to augment other soil bioengineering applications like live fascines, or to stake down straw mulch erosion control blankets. See Figure 10.6. The stakes can be used on their own to provide a simple method of promoting quick growth particularly in a wet area of a slope. Live stakes can be used to stabilize stream banks, where they act to slow the near bank velocities, collect debris, and anchor the surface with a dense root mat.

Figure 10.6: Live Stakes Used to Stabilize a Bar on Laurel Creek

Installation Guidelines Live stakes are gently tapped into the ground at right angles to the bank. An iron bar can be used to form a pilot hole in stiff soils. Stakes are typically installed 0.6 m to 0.9 m apart in a triangular pattern. See Figure 10.7. Stakes split during installation should be pulled out and replaced. A shot filled hammer can be used to minimize damage to the stake during installation. It is important that soil is well packed around the stake to minimize contact between plant material and air spaces. This is achieved when the installer tamps around the stake with his/her boot. Typically, only 20% of the stake length extends from the slope.

Figure 10.7: Schematic Diagram of Live Stake Installation



Riprap with Brush Layer

Description This technique consists of installing brush layers between successive layers of soil that are faced with riprap.

Applications This technique is used for stream bank protection in areas of high velocity. See Figure 10.8. The riprap protects the bank from flow velocities, while the roots from the brush layer form a matted layer behind the riprap. Also, the vegetation that extends from the riprap serves to trap sediment and provide shade for the stream. This is a good method for becoming familiar with soil bioengineering, since it offers the safety factor of a traditional erosion control method like riprap, while giving the practitioner exposure to brush layering techniques.

Installation Guidelines The installation is very similar to the brush layer installation, however the exposed soil layer is covered with riprap.

Riprap with Live Staking

Description This application consists of live stakes installed in the spaces among riprap.

Application This application is similar to riprap with brush layers, however it can be applied in areas with existing riprap in place. Vigorous top growth can provide shade for a stream, making the system suitable for stream bank protection. Other benefits include the formation of a strong root mat behind the riprap layer. Before carrying out a live staking project on an area presently protected with riprap it is important to examine the impact on flows. The increase in surface roughness of heavy woody vegetation may be sufficient to change the flooding characteristics in the channel. This is particularly important on smaller channels.

Installation Guidelines Live stakes are installed in the spaces among the riprap in a similar manner to the live stake description. It is important that the stakes extend through the layer of riprap into the underlying soil. If geotextile is present under the riprap it will be necessary to puncture the fabric to install the live stake.

Figure 10.8: Riprap Installation with Brush Layers

Live Cribwall

Description A live cribwall is essentially a cribwall with layers of brush in the spaces between the logs. The log structure deteriorates with time, and the remaining dense root structure continues to stabilize the slope.

Applications Live cribwalls (Figures 10.9 to 10.12) are typically installed to stabilize stream banks in areas of high velocity. The plant material knits the cribwall together and provides shade for the stream. For the underwater portion of the cribwall, dead material is used to reduce velocity and trap sediment. Another option on stream bank applications is to fill the lower portion of the crib with stone to allows for a vertical wall face underwater, further diversifying fish habitat. Placing rock on front of the cribwall is another common practice. If the cribwall is used to shore up the lower portion of an embankment, it is critical that soil stability tests and calculations are carried out.

Installation Guidelines The logs in the cribwall are fastened together with long nails or metal bars. Typically the nail or bar must extend through three logs. The lower portion of the cribwall is often built on the bank and lowered into the stream to avoid underwater construction. The pine branches are placed in the crib in an interwoven pattern. After the crib is placed in the water and back-filled to the water level, construction begins on the upper crib. The wall is constructed with alternating layers of brush and logs. Soil must be well compacted around the plant material.

Figure 10.9: Live Cribwall Installation

Figure 10.10: Schematic Diagram of a Live Cribwall Installation

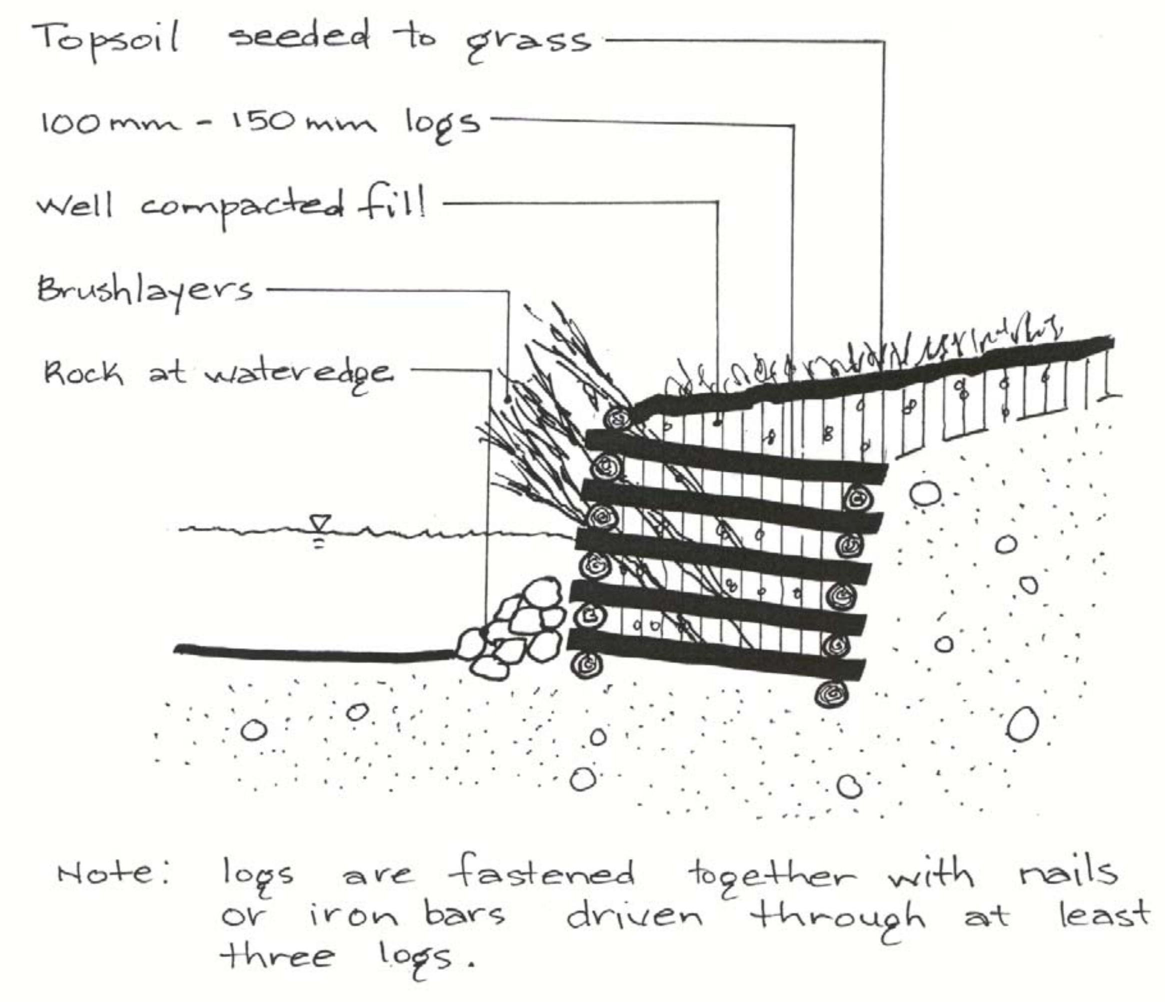


Figure 10.11: Live Cribwall Installations on Small Stream

Figure 10.12: End View of a Live Cribwall

Construction and Material Handling Guidelines

This section gives an overview of the typical plant species used for soil bioengineering projects. In addition, methods for selecting cutting sites and handling live plant material are included.

Plant Species for Soil Bioengineering

Tables 10.13 and 10.14 present information found in Soil Bioengineering for Upland Slope Protection in *Engineering Field Handbook*, USDA (1992). The tables have been modified to include only those plant species common in Ontario. They provide guidelines for selecting plant species. In general, it is preferable to use plants that root easily from cuttings for the soil bioengineering structures in this manual. Rooted stock from species that do not root well from cuttings, may be used to augment the project. The use of rooted stock for the entire project is another option that merits serious consideration, and it is discussed further in the section the Use of Rooted Stock later.

Selecting the Harvest Site

After selecting appropriate plant species, the project team must locate a source for live material. The harvest site should be as close to the project site as possible. Normally, the harvest site should have sufficient access to allow plant materials to be cut and carried to a truck for transport. Sites which have been previously used to harvest cuttings are generally good prospects for suitable material. They usually provide a good supply of rapidly regenerating new shoots within the optimum 12 to 50 mm diameter range. Cuttings from these shoots offer the best potential for root production.

Harvesting Plant Materials

Cuttings are normally harvested with chainsaws, axes or prunners. After cutting the plant, materials are placed in bundles and bound with twine for transport. It is important to note that for optimum results plant material should be cut and installed on the project on the same day, if possible. Since the plant species used for soil bioengineering projects are typically found in wetland type areas, it is important to check with the Ministry of Natural Resources and the local conservation authority to determine if there are any constraints to entering the wetland to carry out the harvest. Typical concerns could include impacts on migratory species, as well as local plant and animal species.

Table 10.13: Plant Tolerance

Name	Deposition Tolerance	Flooding Tolerance	Drought Tolerance	Salt Tolerance
Betula papyrifera Paper birch	Medium	Medium	Medium	Medium
Cornus racemosa Gray dogwood	Medium	Medium	High	Low
Cornus sericea *ssp. stolonifera* Red osier dogwood	Low	High	Medium	Low
Crataegus Sp. Hawthorn	Medium	Low	High	Low
Physocarpus opulifolius Common ninebark	Low	Medium	Medium	Medium
Populus deltoides Eastern cottonwood	Medium	High	Medium	Low
Robina pseudoacacia Black locust	Medium	Low	High	High
Rubus strigosus Red raspberry	Medium	Low	Medium	Low
ssp. interior Sandbar willow	High	High	Low	High
Salix bonplandiana Pussy willow	Medium	Medium	Low	---
Salix nigra Black willow	High	High	Medium	Medium
Sambucus canadensis American elderberry	High	Medium	Medium	Low
Sambucus racemosa Red elderberry	Medium	Low	Medium	Low
Viburnum lentago Nannyberry viburnum	Medium	Low	Medium	Low

Table 10.14: Soil Bioengineering Plant Species

Name	Habitat Value	Size/Form	Root Type	Rooting Ability from Cuttings
Betula papyrifera Paper birch	Good	Tree	Fibrous shallow	Poor
Cornus racemosa Gray dogwood	Very good	Medium/small shrub	Shallow	Good
Cornus sericea ssp. stolonifera Red osier dogwood	Very good	Medium/small shrub	shallow	Very good
Crataegus Sp. Hawthorn	Good	Small dense tree	Tap root	Fair
Physocarpus opulifolius Common ninebark	Good	Medium/high shrub	Shallow lateral	Fair/good
Populus deltoides Eastern cottonwood	Good	Large tree	Shallow	Very good
Robina seudoacacia Black locust	Very good	Tree	Shallow	Good
Rubus strigosus Red raspberry	Very good	Small shrub	Fibrous	Good
ssp. interior Sandbar willow	Good	Large shrub	Shallow to deep	Fair/good
Salix bonplandiana Pussy willow	Good	Medium shrub	Fibrous	Very good
Salix nigra Black willow	Good	Large shrub/ small tree	Shallow to deep	Excellent
Sambucus canadensis American elderberry	Very good	Medium shrub	Fibrous	Good
Sambucus racemosa Red elderberry	Good	Medium shrub	---	Good
Viburnum lentago Nannyberry viburnum	Good	Large shrub	Shallow	Fair/good

Transporting the Live Material

The bundles of cuttings should be transported to the project site as soon as feasible after cutting. It is important to cover the material during transport to minimize the drying effect on the cuttings. If necessary, live materials can be stored on site in a pond over night, although immediate installation after harvest is preferred. There have been reports of beavers removing material overnight.

Soil Conditions

With any soil bioengineering system, soil fertility is critical for success. Topsoil provides a healthy growing medium if available. Since a high degree of contact between soil and plant material is required, uniformly graded gravels are not recommended, as there may be voids that are difficult to fill. It tends to be difficult to achieve sufficient compaction when using wet clay soil.

Construction

It is important that construction is overseen by someone familiar with soil bioengineering techniques, particularly for the more complicated structures.

The Use of Rooted Stock

This section is drawn from written correspondence from Tribble (1995). An alternative approach to using live cuttings is the use of rooted stock. A number of commercial growers provide the appropriate plant materials. The use of rooted stock offers the following advantages:

- Higher survivability and growth rate can be expected.
- A wider range of species can be employed within the planting scheme, since species that are more difficult to root under field conditions would not be ruled out. Greater species diversity would allow the planting group to survive larger fluctuations in growing stresses and would add to its visual and environmental diversity.
- There would be a guaranteed source and quantity of material contracted prior to use.
- The material could be held and shipped from cold storage in damp sawdust to accommodate delays due to weather or construction progress.
- There would be a reduction in potential habitat disruption which can occur when harvesting crews are sent into wetland areas during the spring.
- Sizeable orders of rooted native stock would foster this novel aspect within the nursery trade, making future supply more secure.
- Pre-rooted nursery stock may be fairly cost competitive when the comparison is based on the number of successfully established plants.

Practical Experience with Soil Bioengineering

All photographs in this chapter have been taken within the Grand River watershed in southwestern Ontario. Project sponsors within the watershed have included the Cities of Kitchener, Waterloo, Cambridge, and the Grand River Conservation Authority. Generally, the experience with soil bioengineering has been positive. All of the techniques described in this chapter have been used to quickly establish woody vegetation on stream banks.

Noteworthy observations include:

- Material harvested and installed on the same day appears to have a higher rate of survival than material that is harvested, stored and then put in the ground.
- The surrounding soil tends to be washed away from live fascines that run parallel to the flow, however well staked fascines stay in place and continue to prevent bank erosion for sufficient time to allow for the establishment of vegetation on the stream bank behind the fascine.
- If discovered by beavers, the willow and dogwood plants in a soil bioengineering structure provide an excellent food source. After being trimmed, the plants grow back with renewed vigour.
- It is important that contractors bidding on soil bioengineering projects are familiar with handling live plant materials.
- A field orientation session, led by an experienced practitioner, is important to the success of a project.
- The use of rooted stock tends to increase the percentage of surviving plants in a soil bioengineering structure.
- Willow and dogwood cuttings will grow well on poor quality, low nutrient soils, provided they have adequate moisture.
- Live stakes split during installation have a very low survival rate.
- A microclimate favourable for the establishment of grasses tends to be established on the slope area between brush layers.

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Ministry
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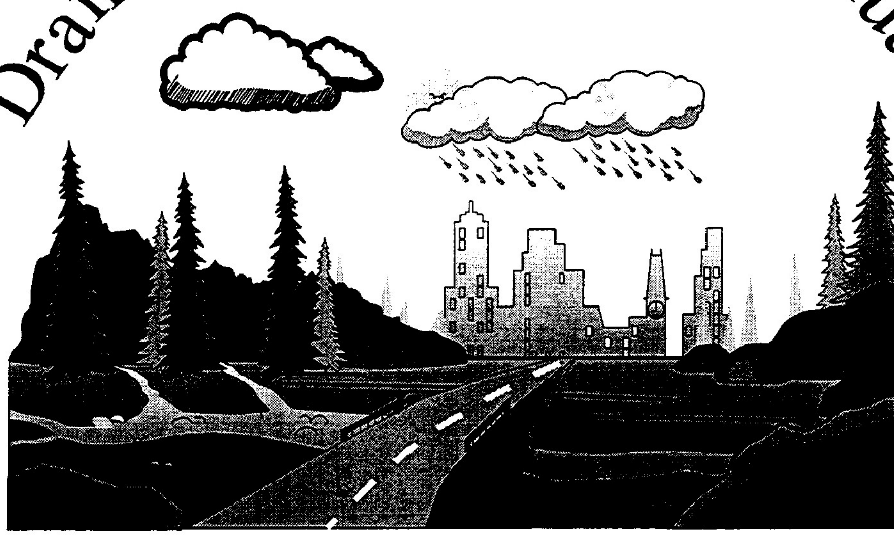
Part 4



Drainage and Hydrology Section
Transportation Engineering Branch
Quality and Standards Division

Summary List of Design Charts

List of Design Charts

Design Chart 1.01(a) : District IDF Curves

District 1 - Chatham

Applicable to basins north of and including Dresden.

INTENSITY - DURATION - FREQUENCY VALUES

Duration (min)	Return Period (years) 2 (mm/hr)	5 (mm/hr)	10 (mm/hr)	25 (mm/hr)	50 (mm/hr)	100 (mm/hr)
5	120	165	200	240	270	300
10	84	115	130	255	175	195
15	67	94	110	135	150	170
30	41	58	69	84	94	105
60	26	38	46	56	63	70
120	13	19	23	28	32	36
360	6.3	8.6	10	11	13	15
720	3.6	4.6	5.3	6.1	6.7	7.3
1440	2.1	2.6	2.9	3.4	3.7	4.0

Applicable to basins south of Dresden.

INTENSITY - DURATION - FREQUENCY VALUES

Duration (min)	Return Period (years) 2 (mm/hr)	5 (mm/hr)	10 (mm/hr)	25 (mm/hr)	50 (mm/hr)	100 (mm/hr)
5	110	140	165	190	210	235
10	73	97	115	135	150	165
15	60	80	93	110	120	135
30	40	56	67	81	91	100
60	26	36	43	51	58	64
120	19	26	30	35	39	43
360	6.3	8.7	11	12	13	14
720	3.9	5.3	6.2	7.4	8.2	9.1
1440	2.1	2.8	3.2	3.7	4.1	4.5

Source: MTO (1989)

Design Chart 1.01(b) : District IDF Curves

District 2 - London

Applicable to basins west of and including London.

INTENSITY - DURATION - FREQUENCY VALUES

	Return Period (years)					
Duration (min)	2 (mm/hr)	5 (mm/hr)	10 (mm/hr)	25 (mm/hr)	50 (mm/hr)	100 (mm/hr)
5	100	140	170	205	230	255
10	74	110	130	155	175	190
15	64	87	105	125	140	150
30	40	56	65	81	91	100
60	25	36	43	52	59	66
120	13	19	23	27	31	35
360	6.4	8.6	10	12	13	14
720	3.7	4.8	5.5	6.3	7.0	7.6
1440	2.1	2.7	3.1	3.6	4.0	4.3

Applicable to basins east of London.

INTENSITY - DURATION - FREQUENCY VALUES

	Return Period (years)					
Duration (min)	2 (mm/hr)	5 (mm/hr)	10 (mm/hr)	25 (mm/hr)	50 (mm/hr)	100 (mm/hr)
5	100	135	155	185	205	230
10	73	100	120	140	160	175
15	60	76	88	100	110	120
30	39	51	59	69	77	84
60	24	32	37	44	48	53
120	12	17	20	24	27	29
360	6.2	8.4	9.9	12	13	14
720	3.7	4.9	5.7	6.6	7.4	8.1
1440	2.1	2.7	3.1	3.7	4.0	4.4

Source: MTO (1989)

Design Chart 1.01(c) : District IDF Curves

District 3 - Stratford

Applicable to basins east of and including Waterloo.

INTENSITY - DURATION - FREQUENCY VALUES

Duration (min)	Return Period (years) 2 (mm/hr)	5 (mm/hr)	10 (mm/hr)	25 (mm/hr)	50 (mm/hr)	100 (mm/hr)
5	100	135	155	180	200	220
10	74	95	110	125	140	150
15	64	83	95	110	125	135
30	40	54	64	76	85	94
60	25	36	43	52	59	66
120	13	20	23	28	32	34
360	6.6	9.0	11	13	14	15
720	3.7	5.0	5.8	6.9	7.7	8.5
1440	2.2	2.9	3.3	3.9	4.3	4.7

Applicable to basins west of Waterloo.

INTENSITY - DURATION - FREQUENCY VALUES

Duration (min)	Return Period (years) 2 (mm/hr)	5 (mm/hr)	10 (mm/hr)	25 (mm/hr)	50 (mm/hr)	100 (mm/hr)
5	97	130	150	175	195	215
10	76	94	105	120	130	140
15	60	74	84	96	105	115
30	40	52	60	70	77	84
60	25	33	39	46	51	56
120	14	20	24	29	33	37
360	6.9	9.4	11	13	15	16
720	3.8	5.1	6.0	7.0	7.8	8.6
1440	2.2	2.8	3.2	3.7	4.1	4.5

Source: MTO (1989)

Design Chart 1.01(d) : District IDF Curves

District 4 - Burlington

Applicable to basins northeast of and including Highway No. 6.

INTENSITY - DURATION - FREQUENCY VALUES

	Return Period (years)					
Duration (min)	2 (mm/hr)	5 (mm/hr)	10 (mm/hr)	25 (mm/hr)	50 (mm/hr)	100 (mm/hr)
5	100	135	155	180	200	220
10	74	96	110	130	140	155
15	62	82	96	115	125	140
30	39	52	61	72	80	88
60	23	33	40	48	54	60
120	13	18	21	25	28	31
360	5.9	8.1	9.6	11	13	14
720	3.8	4.9	5.7	6.7	7.4	8.1
1440	2.2	2.8	3.2	3.8	4.2	4.5

Applicable to basins southwest of Highway No. 6.

INTENSITY - DURATION - FREQUENCY VALUES

	Return Period (years)					
Duration (min)	2 (mm/hr)	5 (mm/hr)	10 (mm/hr)	25 (mm/hr)	50 (mm/hr)	100 (mm/hr)
5	97	130	150	180	195	215
10	69	91	105	125	140	150
15	57	74	85	98	110	120
30	36	47	55	64	71	78
60	23	30	35	41	45	50
120	12	16	19	22	25	27
360	6.0	8.2	9.6	11	13	14
720	3.5	4.7	5.4	6.4	7.2	7.8
1440	2.0	2.6	3.1	3.6	4.0	4.4

Source: MTO (1989)

Design Chart 1.01(e) : District IDF Curves

District 5 - Owen Sound

Applicable to basins west of and including Collingwood.

INTENSITY - DURATION - FREQUENCY VALUES

	Return Period (years)					
Duration (min)	2 (mm/hr)	5 (mm/hr)	10 (mm/hr)	25 (mm/hr)	50 (mm/hr)	100 (mm/hr)
5	98	130	150	180	200	215
10	70	90	105	120	130	145
15	58	77	89	105	115	125
30	38	50	57	67	74	82
60	25	33	38	46	51	56
120	17	21	24	28	31	34
360	6.3	8.8	11	13	14	16
720	3.5	4.7	5.5	6.5	7.2	8.0
1440	1.9	2.6	3.0	3.6	4.0	4.5

Applicable to basins east of Collingwood.

INTENSITY - DURATION - FREQUENCY VALUES

	Return Period (years)					
Duration (min)	2 (mm/hr)	5 (mm/hr)	10 (mm/hr)	25 (mm/hr)	50 (mm/hr)	100 (mm/hr)
5	95	125	150	175	195	215
10	79	100	115	130	145	160
15	62	82	95	110	125	135
30	40	53	62	73	81	89
60	25	34	41	49	55	61
120	14	19	23	27	30	33
360	6.1	8.4	10	12	13	15
720	3.3	4.6	5.5	6.6	7.4	8.2
1440	2.1	2.7	3.1	3.6	4.0	4.4

Source: MTO (1989)

Design Chart 1.01(f) : District IDF Curves

District 6 - Toronto

Applicable to basins west of and including Toronto.

INTENSITY - DURATION - FREQUENCY VALUES

	Return Period (years)					
Duration (min)	2 (mm/hr)	5 (mm/hr)	10 (mm/hr)	25 (mm/hr)	50 (mm/hr)	100 (mm/hr)
5	100	135	155	180	200	220
10	86	105	120	140	150	165
15	63	130	100	120	135	150
30	38	54	63	79	89	99
60	23	33	39	48	54	60
120	14	19	22	27	29	33
360	6.2	8.4	9.8	12	13	14
720	3.6	4.7	5.5	6.4	7.1	7.8
1440	2.1	2.7	3.1	3.7	4.1	4.5

Applicable to basins east of Toronto.

INTENSITY - DURATION - FREQUENCY VALUES

	Return Period (years)					
Duration (min)	2 (mm/hr)	5 (mm/hr)	10 (mm/hr)	25 (mm/hr)	50 (mm/hr)	100 (mm/hr)
5	97	130	150	175	195	215
10	74	94	110	125	135	150
15	59	77	88	105	115	125
30	36	49	57	68	76	84
60	22	30	36	43	49	54
120	13	18	21	25	28	31
360	6.0	8.2	9.7	12	13	14
720	3.3	4.5	5.3	6.3	7.0	7.8
1440	1.8	2.5	2.9	3.4	3.8	4.2

Source: MTO (1989)

Design Chart 1.01(g) : District IDF Curves

District 7 - Port Hope

Applicable to basins north of and including Lindsay.

INTENSITY - DURATION - FREQUENCY VALUES

	Return Period (years)					
Duration (min)	2 (mm/hr)	5 (mm/hr)	10 (mm/hr)	25 (mm/hr)	50 (mm/hr)	100 (mm/hr)
5	88	120	140	165	185	205
10	67	86	98	115	125	140
15	56	72	82	96	105	115
30	36	45	51	59	65	71
60	22	29	34	40	44	48
120	13	18	21	25	27	30
360	6.0	7.8	8.9	10	12	13
720	3.3	4.2	4.9	5.7	6.2	6.8
1440	1.7	2.2	2.5	2.9	3.2	3.5

Applicable to basins south of Lindsay.

INTENSITY - DURATION - FREQUENCY VALUES

	Return Period (years)					
Duration (min)	2 (mm/hr)	5 (mm/hr)	10 (mm/hr)	25 (mm/hr)	50 (mm/hr)	100 (mm/hr)
5	82	115	135	160	180	200
10	58	77	90	105	120	130
15	48	62	72	85	94	105
30	29	37	43	49	54	49
60	19	25	29	33	37	40
120	12	16	18	22	24	26
360	5.6	7.2	8.4	9.8	11	12
720	3.2	4.1	4.8	5.6	6.2	6.7
1440	1.8	2.3	2.6	3.0	3.2	3.5

Source: MTO (1989)

Design Chart 1.01(h) : District IDF Curves

District 8 - Kingston

Applicable to basins east of and including Kingston.

INTENSITY - DURATION - FREQUENCY VALUES

Duration (min)	Return Period (years) 2 (mm/hr)	5 (mm/hr)	10 (mm/hr)	25 (mm/hr)	50 (mm/hr)	100 (mm/hr)
5	96	125	150	175	195	215
10	68	90	105	120	135	150
15	54	71	82	96	105	115
30	35	46	54	64	71	78
60	19	26	31	38	42	46
120	12	16	19	23	25	28
360	5.6	7.1	8.1	9.4	10	11
720	3.4	4.4	5.0	5.9	6.5	7.1
1440	1.9	2.5	2.9	3.4	3.7	4.1

Applicable to basins west of Kingston.

INTENSITY - DURATION - FREQUENCY VALUES

Duration (min)	Return Period (years) 2 (mm/hr)	5 (mm/hr)	10 (mm/hr)	25 (mm/hr)	50 (mm/hr)	100 (mm/hr)
5	79	110	130	160	180	200
10	58	77	90	105	120	130
15	46	62	74	87	98	110
30	31	41	48	56	62	68
60	19	26	30	35	39	43
120	12	16	18	21	24	28
360	5.6	7.0	7.9	9.1	10	11
720	3.2	4.3	4.9	5.8	6.4	7.1
1440	1.8	2.2	2.5	2.9	3.2	3.4

Source: MTO (1989)

Design Chart 1.01(i) : District IDF Curves

District 9 - Kemptville

Applicable to basins west of and including Kemptville.

INTENSITY - DURATION - FREQUENCY VALUES

Duration (min)	Return Period (years) 2 (mm/hr)	5 (mm/hr)	10 (mm/hr)	25 (mm/hr)	50 (mm/hr)	100 (mm/hr)
5	96	130	150	175	195	215
10	71	92	105	125	135	150
15	55	76	90	110	120	135
30	34	47	55	64	72	79
60	20	28	33	39	44	49
120	12	16	19	22	25	27
360	5.8	7.3	8.4	9.7	11	12
720	3.4	4.0	4.5	5.0	5.4	5.8
1440	1.8	2.3	2.6	3.0	3.3	3.6

Applicable to basins east of Kemptville.

INTENSITY - DURATION - FREQUENCY VALUES

Duration (min)	Return Period (years) 2 (mm/hr)	5 (mm/hr)	10 (mm/hr)	25 (mm/hr)	50 (mm/hr)	100 (mm/hr)
5	120	150	170	200	220	240
10	82	105	115	135	150	160
15	60	81	95	115	125	140
30	37	51	60	72	80	89
60	24	32	38	45	50	56
120	12	16	18	23	26	29
360	6.3	8.0	9.2	11	12	13
720	3.5	4.7	5.5	6.5	7.2	8.0
1440	2.0	2.6	3.1	3.6	4.0	4.4

Source: MTO (1989)

Design Chart 1.01(j) : District IDF Curves

District 10 - Bancroft

INTENSITY - DURATION - FREQUENCY VALUES

Duration (min)	Return Period (years) 2 (mm/hr)	5 (mm/hr)	10 (mm/hr)	25 (mm/hr)	50 (mm/hr)	100 (mm/hr)
5	80	115	135	160	180	200
10	62	82	95	110	125	135
15	52	72	84	100	115	125
30	34	46	54	64	71	79
60	20	28	33	39	44	49
120	12	16	19	23	25	28
360	5.7	7.0	7.9	9.0	9.9	11
720	3.2	4.1	4.7	5.4	6.0	6.9
1440	1.7	2.1	2.4	2.7	3.0	3.2

Design Chart 1.01(k) : District IDF Curves

District 11 - Huntsville

INTENSITY - DURATION - FREQUENCY VALUES

Duration (min)	Return Period (years) 2 (mm/hr)	5 (mm/hr)	10 (mm/hr)	25 (mm/hr)	50 (mm/hr)	100 (mm/hr)
5	95	125	150	175	195	215
10	73	94	110	125	140	150
15	57	75	86	100	110	125
30	40	51	59	68	76	82
60	24	32	37	44	49	55
120	14	18	20	24	26	28
360	6.0	7.5	8.6	9.9	11	12
720	3.2	4.1	4.7	5.5	6.0	6.6
1440	1.9	2.4	2.7	3.1	3.4	3.7

Source: MTO (1989)

Design Chart 1.01(l) : District IDF Curves

District 13 - North Bay

INTENSITY - DURATION - FREQUENCY VALUES

	Return Period (years)					
Duration (min)	2 (mm/hr)	5 (mm/hr)	10 (mm/hr)	25 (mm/hr)	50 (mm/hr)	100 (mm/hr)
5	95	125	150	175	195	215
10	73	95	110	130	145	160
15	55	73	85	100	115	125
30	38	49	57	67	74	82
60	23	31	37	44	49	54
120	13	18	21	25	28	31
360	6.1	8.1	9.4	11	12	14
720	3.6	4.5	5.2	5.9	6.5	7.1
1440	2.1	2.7	3.1	3.6	3.9	4.3

Design Chart 1.01(m) : District IDF Curves

District 14 - New Liskeard

INTENSITY - DURATION - FREQUENCY VALUES

	Return Period (years)					
Duration (min)	2 (mm/hr)	5 (mm/hr)	10 (mm/hr)	25 (mm/hr)	50 (mm/hr)	100 (mm/hr)
5	85	115	135	160	180	200
10	58	79	94	110	125	140
15	44	61	72	86	97	110
30	29	41	49	59	67	74
60	18	26	31	37	42	47
120	11	15	17	21	23	25
360	5.4	7.2	8.4	9.9	11	12
720	3.4	4.5	5.3	6.3	7.1	7.8
1440	2.1	2.8	3.2	3.8	4.3	4.7

Source: MTO (1989)

Design Chart 1.01(n) : District IDF Curves

District 16 - Cochrane

Applicable to basins east of and including Kapuspasing.

INTENSITY - DURATION - FREQUENCY VALUES

	Return Period (years)					
Duration (min)	2 (mm/hr)	5 (mm/hr)	10 (mm/hr)	25 (mm/hr)	50 (mm/hr)	100 (mm/hr)
5	85	120	140	165	185	205
10	56	78	93	110	125	140
15	42	61	74	89	100	110
30	28	41	49	60	68	76
60	17	26	31	38	43	48
120	11	16	19	23	25	28
360	5.4	7.6	8.9	11	12	13
720	3.1	4.3	5.1	6.1	6.8	7.6
1440	2.0	2.7	3.1	3.7	4.2	4.6

Applicable to basins west of Kapuspasing.

INTENSITY - DURATION - FREQUENCY VALUES

	Return Period (years)					
Duration (min)	2 (mm/hr)	5 (mm/hr)	10 (mm/hr)	25 (mm/hr)	50 (mm/hr)	100 (mm/hr)
5	78	110	130	160	175	195
10	53	75	91	110	125	140
15	38	56	69	85	96	110
30	22	31	36	43	48	53
60	15	22	27	33	38	42
120	10	14	16	20	22	24
360	4.7	6.6	7.9	9.5	11	12
720	3.1	4.1	4.7	5.6	6.2	6.8
1440	1.6	2.2	2.5	3.0	3.3	3.7

Source: MTO (1989)

Design Chart 1.01(o) : District IDF Curves

District 17 - Sudbury

INTENSITY - DURATION - FREQUENCY VALUES

Duration (min)	Return Period (years) 2 (mm/hr)	5 (mm/hr)	10 (mm/hr)	25 (mm/hr)	50 (mm/hr)	100 (mm/hr)
5	85	115	140	165	185	205
10	68	89	105	120	135	150
15	51	70	83	99	110	125
30	33	45	53	63	70	77
60	21	28	33	39	44	48
120	12	16	19	22	25	27
360	5.5	7.4	8.6	10	11	13
720	3.4	4.4	5.1	5.9	6.6	7.2
1440	2.1	2.6	3.0	3.5	3.8	4.2

Design Chart 1.01(p) : District IDF Curves

District 18 - Saulte Ste. Marie

INTENSITY - DURATION - FREQUENCY VALUES

Duration (min)	Return Period (years) 2 (mm/hr)	5 (mm/hr)	10 (mm/hr)	25 (mm/hr)	50 (mm/hr)	100 (mm/hr)
5	80	110	130	155	175	195
10	54	73	86	100	115	125
15	41	56	66	78	88	97
30	28	38	45	53	60	66
60	18	24	29	34	38	41
120	12	16	18	21	24	26
360	4.9	7.0	8.4	10	11	13
720	3.2	4.3	5.1	6.0	6.7	7.5
1440	1.8	2.4	2.9	3.4	3.9	4.3

Source: MTO (1989)

Design Chart 1.01(q) : District IDF Curves

District 19 - Thunder Bay

Applicable to basins west of and including Red Rock.

INTENSITY - DURATION - FREQUENCY VALUES

Duration (min)	Return Period (years) 2 (mm/hr)	5 (mm/hr)	10 (mm/hr)	25 (mm/hr)	50 (mm/hr)	100 (mm/hr)
5	100	135	155	185	205	225
10	71	96	110	135	150	165
15	56	79	94	115	130	140
30	35	49	59	71	80	88
60	22	30	36	43	48	53
120	13	18	21	25	28	31
360	5.7	7.4	8.6	10	11	12
720	3.3	4.2	4.9	5.7	6.3	6.8
1440	2.0	2.6	3.0	3.5	3.9	4.2

Applicable to basins east of Red Rock.

INTENSITY - DURATION - FREQUENCY VALUES

Duration (min)	Return Period (years) 2 (mm/hr)	5 (mm/hr)	10 (mm/hr)	25 (mm/hr)	50 (mm/hr)	100 (mm/hr)
5	84	120	140	165	185	205
10	58	82	97	115	130	145
15	47	66	80	96	110	120
30	29	40	48	57	64	71
60	18	24	28	33	37	41
120	11	14	17	20	22	24
360	4.8	6.4	7.4	8.7	9.6	11
720	3.2	4.0	4.6	5.2	5.8	6.3
1440	1.8	2.3	2.7	3.2	3.6	3.9

Source: MTO (1989)

Design Chart 1.01(r) : District IDF Curves

District 20 - Kenora

Applicable to basins north of and including Dryden.

INTENSITY - DURATION - FREQUENCY VALUES

Duration (min)	Return Period (years) 2 (mm/hr)	5 (mm/hr)	10 (mm/hr)	25 (mm/hr)	50 (mm/hr)	100 (mm/hr)
5	95	135	160	195	220	245
10	70	95	110	130	145	160
15	49	70	83	100	115	125
30	34	46	55	66	74	82
60	25	33	39	45	50	55
120	12	17	20	24	27	30
360	5.5	7.4	8.6	10	11	12.5
720	3.3	4.2	4.9	5.7	6.3	6.9
1440	1.8	2.4	2.8	3.2	3.6	4.0

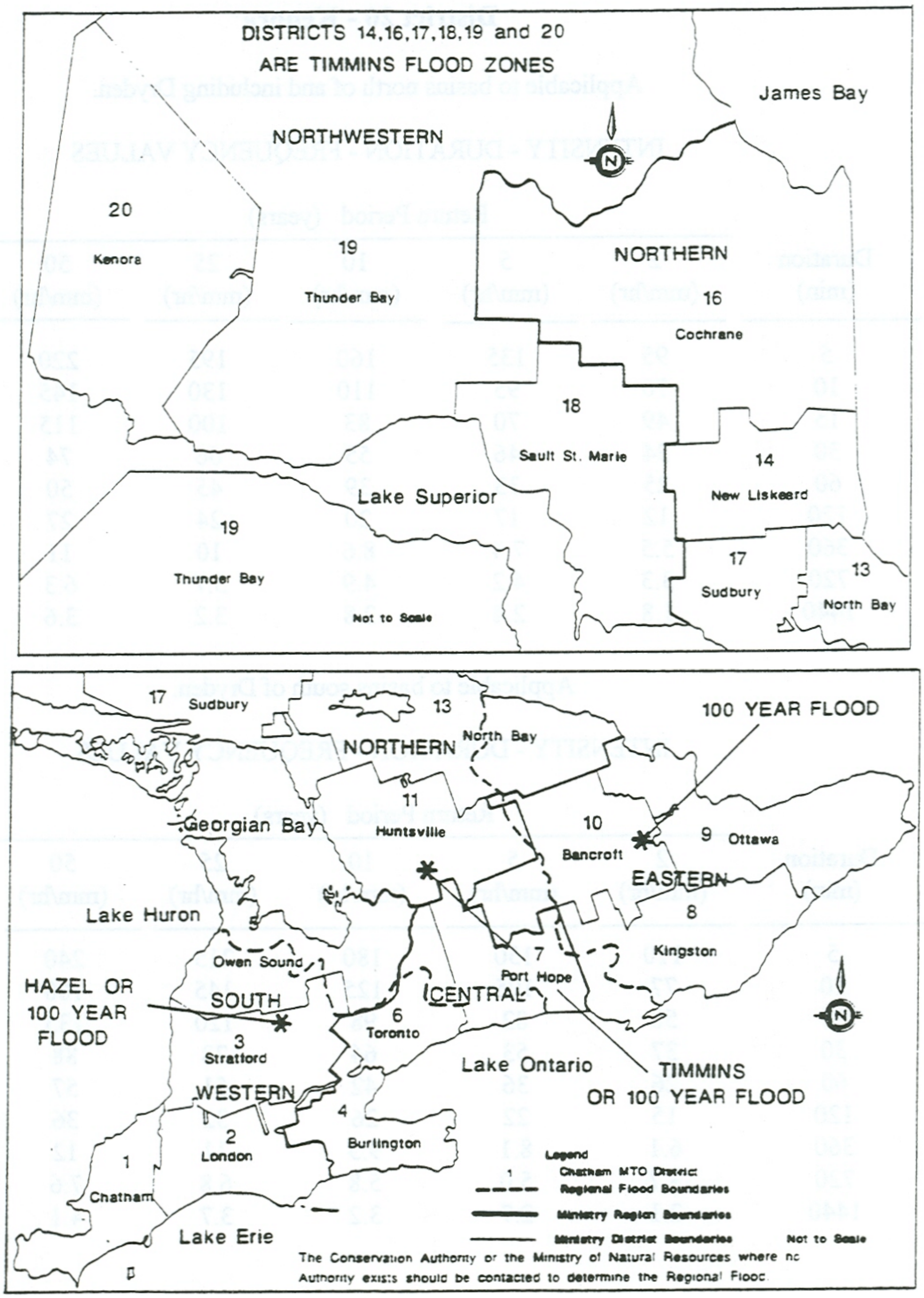
Applicable to basins south of Dryden.

INTENSITY - DURATION - FREQUENCY VALUES

Duration (min)	Return Period (years) 2 (mm/hr)	5 (mm/hr)	10 (mm/hr)	25 (mm/hr)	50 (mm/hr)	100 (mm/hr)
5	110	150	180	215	240	265
10	77	105	125	145	160	180
15	58	82	98	120	135	150
30	37	53	64	78	88	99
60	26	36	42	51	57	63
120	15	22	26	32	36	40
360	6.1	8.1	9.5	11	12	14
720	3.7	5.0	5.8	6.8	7.6	8.4
1440	2.1	2.7	3.2	3.7	4.1	4.5

Source: MTO (1989)

Design Chart 1.02: Spatial Extent of Zones for Application of Regional Storms



Source: MTO (1985)

Design Chart 1.03: Hurricane Hazel

	Depth		Percent of 12 hour
	(mm)	(inches)	
First 36 hours	73	2.90	
37th hour	6	.25	3
38th hour	4	.17	2
39th hour	6	.25	3
40th hour	13	.50	6
41st hour	17	.66	8
42nd hour	13	.50	6
43rd hour	23	.91	11
44th hour	13	.50	6
45th hour	13	.50	6
46th hour	53	2.08	25
47th hour	38	1.49	18
48th hour	13	.50	6
	285	11.21	100

Drainage Area (km^2)	Percentage
0 to 25	100.0
26 to 45	99.2
46 to 65	98.2
66 to 90	97.1
91 to 115	96.3
116 to 140	95.4
141 to 165	94.8
166 to 195	94.2
196 to 220	93.5
221 to 245	92.7
246 to 270	92.0
271 to 450	89.4
451 to 575	86.7
576 to 700	84.0
701 to 850	82.4
851 to 1000	80.8
1001 to 1200	79.3
1201 to 1500	76.6
1501 to 1700	74.4
1701 to 2000	73.3
2001 to 2200	71.7
2201 to 2500	70.2
2501 to 2700	69.0
2701 to 4500	64.4
4501 to 6000	61.4
6001 to 7000	58.9
7001 to 8000	57.4

Source: Ministry of Transportation, MTO (1989)

Design Chart 1.04: Timmins Storm

	Depth		Percent of 12 hour
	(mm)	(inches)	
1st hour	15	0.6	8
2nd hour	20	0.8	10
3rd hour	10	0.4	6
4th hour	3	0.1	1
5th hour	5	0.2	3
6th hour	20	0.8	10
7th hour	43	1.7	23
8th hour	20	0.8	10
9th hour	23	0.9	12
10th hour	13	0.5	6
11th hour	13	0.5	7
12th hour	8	0.3	4
	193	7.6	100

Drainage Area (km^2)	Percentage
0 to 25	100.0
26 to 50	97
51 to 75	94
76 to 100	90
101 to 150	87
151 to 200	84
201 to 250	82
251 to 375	79
376 to 500	76
501 to 750	74
751 to 1000	70
1001 to 1250	68
1251 to 1500	66
1501 to 1800	65
1801 to 2100	64
2101 to 2300	63
2301 to 2600	62
2601 to 3900	58
3901 to 5200	56
5201 to 6500	53
6501 to 8000	50

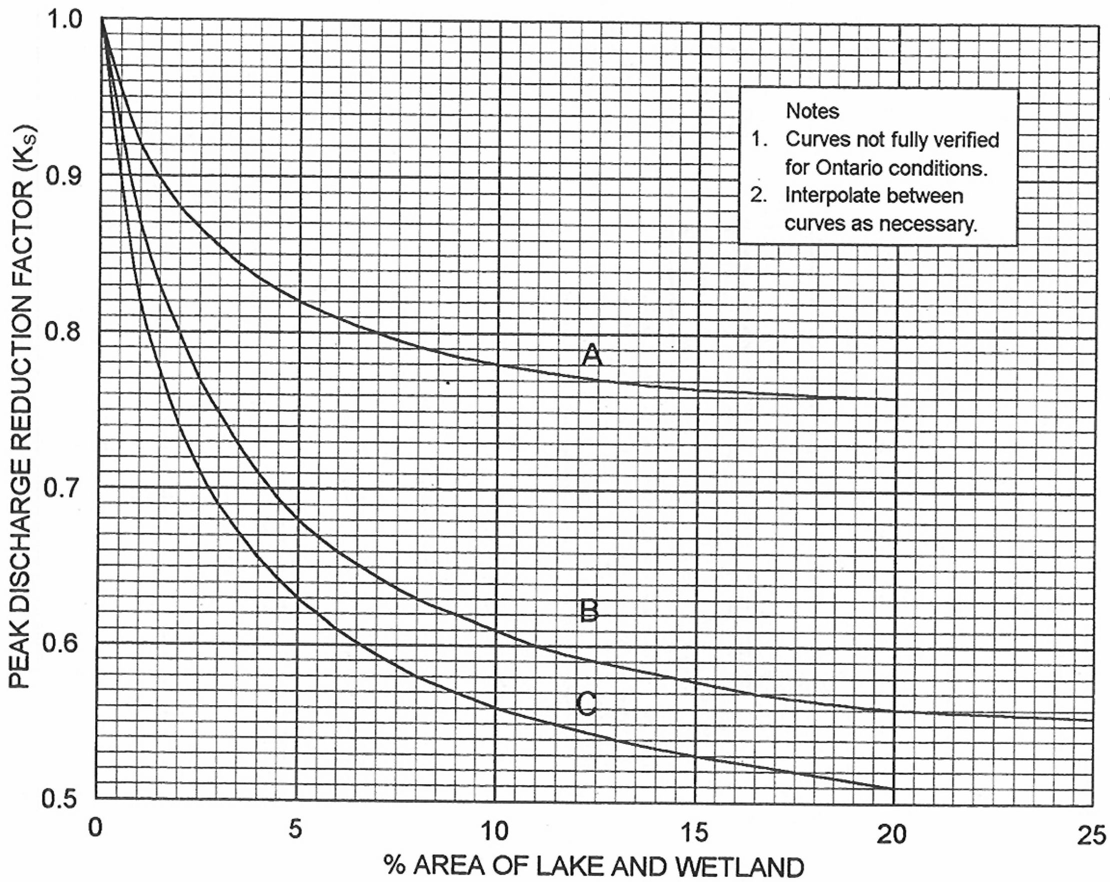
Source: Ministry of Transportation, MTO (1989)

Design Chart 1.05: SCS Type II Distribution

6 hour			12 hour			24 hour		
Time end'g, hour	F_{inc} (%)	F_{cum} (%)	Time end'g, hour	F_{inc} (%)	F_{cum} (%)	Time end'g, hour	F_{inc} (%)	F_{cum} (%)
0	0	0	0	0	0	0	0	0
0.5	2	2	2	5	5	2	2.2	2.2
1	3	5	3	3	8	4	2.6	4.8
1.5	3	8	3.5	2	10	6	3.2	8.0
2	5	13	4	2	12	7	-	-
2.5	6	19	4.5	3	15	8	4.0	12.0
2.75	15	34	5	4	19	8.5	-	-
3	39	73	5.5	6	25	9	2.7	14.7
3.5	11	84	5.75	12	37	9.5	1.6	16.3
4	5	89	6	33	70	9.75	-	-
4.5	4	93	6.5	9	79	10	1.8	18.1
5	3	96	7	4	83	10.5	2.3	20.4
6	4	100	7.5	3	86	11	3.1	23.5
			8	3	89	11.5	4.8	28.3
			10	7	96	11.75	10.4	38.7
			12	4	100	12	27.6	66.3
						12.5	7.2	73.5
						13	3.7	77.2
						13.5	0.7	77.9
						14	4.1	82.0
						16	6.0	88.0
						20	7.2	95.2
						24	4.8	100

Source: Ministry of Natural Resources - MNR (1986)

Design Chart 1.06: Peak Discharge Reduction Factor to Allow for Storage



Curve A - Significant portion of flow passes through detention areas in upper reaches, or elsewhere in basin not in path of flow.

Curve B - Significant portion of flow passes through detention areas distributed throughout basin or in the middle reaches only.

Curve C - Most of detention is located in path of flow at lower end of basin.

Design Chart 1.07: Runoff Coefficients

- Urban for 5 to 10-Year Storms

Land Use	Runoff Coefficient	
	Min.	Max.
Pavement - asphalt or concrete	0.80	0.95
- brick	0.70	0.85
Gravel roads and shoulders	0.40	0.60
Roofs	0.70	0.95
Business - downtown	0.70	0.95
- neighbourhood	0.50	0.70
- light	0.50	0.80
- heavy	0.60	0.90
Residential - single family urban	0.30	0.50
- multiple, detached	0.40	0.60
- multiple, attached	0.60	0.75
- suburban	0.25	0.40
Industrial - light	0.50	0.80
- heavy	0.60	0.90
Apartments	0.50	0.70
Parks, cemeteries	0.10	0.25
Playgrounds (unpaved)	0.20	0.35
Railroad yards	0.20	0.35
Unimproved areas	0.10	0.30
Lawns - Sandy soil		
- flat, to 2%	0.05	0.10
- average, 2 to 7%	0.10	0.15
- steep, over 7%	0.15	0.20
- Clayey soil		
- flat, to 2%	0.13	0.17
- average, 2 to 7%	0.18	0.22
- steep, over 7%	0.25	0.35

For flat or permeable surfaces, use the lower values. For steeper or more impervious surfaces, use the higher values. For return period of more than 10 years, increase above values as 25-year - add 10%, 50-year - add 20%, 100-year - add 25%.

The coefficients listed above are for unfrozen ground.

Design Chart 1.07: Runoff Coefficients (Continued)

- Rural

Land Use & Topography[3]	Soil Texture		
	Open Sand Loam	Loam or Silt Loam	Clay Loam or Clay
CULTIVATED			
Flat 0 - 5% Slopes	0.22	0.35	0.55
Rolling 5 - 10% Slopes	0.30	0.45	0.60
Hilly 10- 30% Slopes	0.40	0.65	0.70
PASTURE			
Flat 0 - 5% Slopes	0.10	0.28	0.40
Rolling 5 - 10% Slopes	0.15	0.35	0.45
Hilly 10- 30% Slopes	0.22	0.40	0.55
WOODLAND OR CUTOVER			
Flat 0 - 5% Slopes	0.08	0.25	0.35
Rolling 5 - 10% Slopes	0.12	0.30	0.42
Hilly 10- 30% Slopes	0.18	0.35	0.52
BARE ROCK	COVERAGE[3]		
	30%	50%	70%
Flat 0 - 5% Slopes	0.40	0.55	0.75
Rolling 5 - 10% Slopes	0.50	0.65	0.80
Hilly 10- 30% Slopes	0.55	0.70	0.85
LAKES AND WETLANDS	0.05		

[2] Terrain Slopes

[3] Interpolate for other values of % imperviousness

Sources: American Society of Civil Engineers - ASCE (1960)
U.S. Department of Agriculture (1972)

Design Chart 1.08: Hydrologic Soil Groups

- Based on Surficial Geology Maps

Map Ref.No.	Soil Type or Texture	Hydrologic Soil Group (Tentative)
	Ground Moraine	
1a	Usually sandy till, stony, varying depth. (Most widespread type in Shield).	Usually B (shallow); may be A or AB
1b	Clayey till, varying depth.	BC-C
	End or Interlobate Moraine	
2a	Sand & stones, deep. (May be rough topography).	A
2b	Sand & stones capped by till, deep.	A-C depending on type of till.
2c	Sand & stones, deep. (Smoother topography).	A
	Kames & Eskers	
3a	Sand & stones, deep. (May be rough topography).	A
3b	Sand & stones capped by till, deep.	A-C depending on type of till.
3c	Sand & stones, deep. (Smoother topography).	A
	Lacustrine	
4a	Clay & silt, in lowlands.	BC-C
4b	Fine sand, in lowlands.	AB-B
4c	Sand, in lowlands.	AB
4d	Sand (deltas & valley trains).	A-AB
	Outwash	
5	Sand, some gravel, deep.	A
	Aeolian	
6	Very fine sand & silt, shallow. (Loess)	B
	Bedrock	
7	Bare bedrock (normally negligible areas).	Varies according to rock type.

Source: Ministry of Natural Resources - MNR

Design Chart 1.08: Hydrologic Soil Groups (Continued)

- Based on Soil Texture

Sands, Sandy Loams and Gravels	
- overlying sand, gravel or limestone bedrock, very well drained	A
- ditto, imperfectly drained	AB
- shallow, overlying Precambrian bedrock or clay subsoil	B
Medium to Coarse Loams	
- overlying sand, gravel or limestone, well drained	AB
- shallow, overlying Precambrian bedrock or clay subsoil	B
Medium Textured Loams	
- shallow, overlying limestone bedrock	B
- overlying medium textured subsoil	BC
Silt Loams, Some Loams	
- with good internal drainage	BC
- with slow internal drainage and good external drainage	C
Clays, Clay Loams, Silty Clay Loams	
- with good internal drainage	C
- with imperfect or poor external drainage	C
- with slow internal drainage and good external drainage	D

Source: U.S. Department of Agriculture (1972)

Design Chart 1.09: Soil/Land Use Curve Numbers

Land Use	Treatment or Practice	Hydrologic Condition[4]	Hydrologic Soil Group			
			A	B	C	D
Fallow	Straight row	---	77	86	91	94
Row crops	"	Poor	72	81	88	91
	"	Good	67	78	85	89
	Contoured	Poor	70	79	84	88
	"	Good	65	75	82	86
	" and terraced	Poor	66	74	8	82
	" " "	Good	62	71	78	81
Small grain	Straight row	Poor	65	76	84	88
		Good	63	75	83	87
	Contoured	Poor	63	74	82	85
		Good	61	73	81	84
	" and terraced	Poor	61	72	79	82
		Good	59	70	78	81
Close-seeded	Straight row	Poor	66	77	85	89
legumes[2]	" "	Good	58	72	81	85
or	Contoured	Poor	64	75	83	85
rotation	"	Good	55	69	78	83
meadow	" and terraced	Poor	63	73	80	83
	" and terraced	Good	51	67	76	80
Pasture		Poor	68	79	86	89
or range		Fair	49	69	79	84
	Contoured	Good	39	61	74	80
	"	Poor	47	67	81	88
	"	Fair	25	59	75	83
		Good	6	35	70	79
Meadow		Good	30	58	71	78
Woods		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	25	55	70	77
Farmsteads		---	59	74	82	86
		---	72	82	87	89
		---	74	84	90	92

For average anticedent soil moisture condition (AMC II)
[2] Close-drilled or broadcast.

[4] The hydrologic condition of cropland is good if a good crop rotation practice is used; it is poor if one crop is grown continuously.

Source: U.S. Department of Agriculture (1972)

Design Chart 1.09: Soil Conservation Service Curve Numbers (Continued)

Land Use or Surface	Hydrologic Soil Group						
	A	AB	B	BC	C	CD	D
Fallow (special cases only)	77	82	86	89	91	93	94
Crop and other improved land	66** (62)	70** (68)	74	78	82	84	86 AMC I
Pasture & other unimproved land	58* (38)	62* (51)	65	71	76	79	81
Woodlots and forest	50* (30)	54* (44)	58	65	71	74	77
Impervious areas (paved)							98
Bare bedrock draining directly to stream by surface flow							98
Bare bedrock draining indirectly to stream as groundwater (usual case)							70
Lakes and wetlands							50

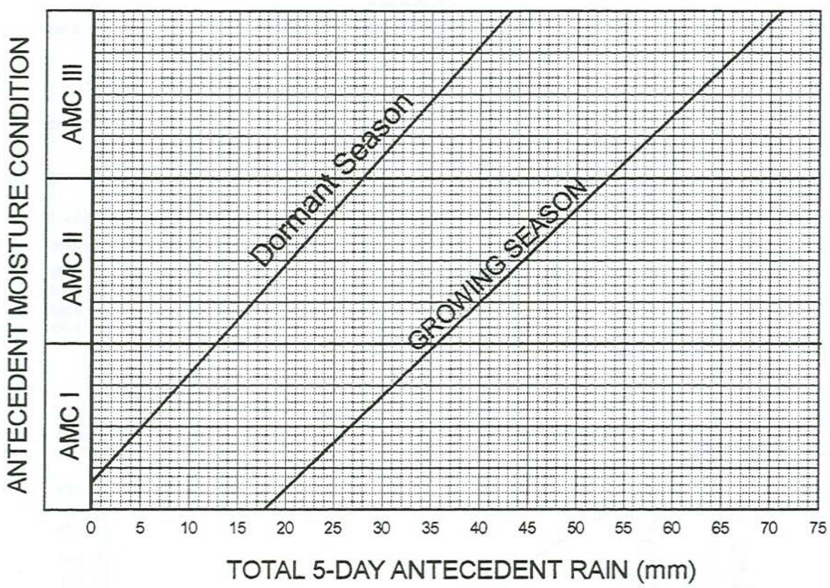
Notes

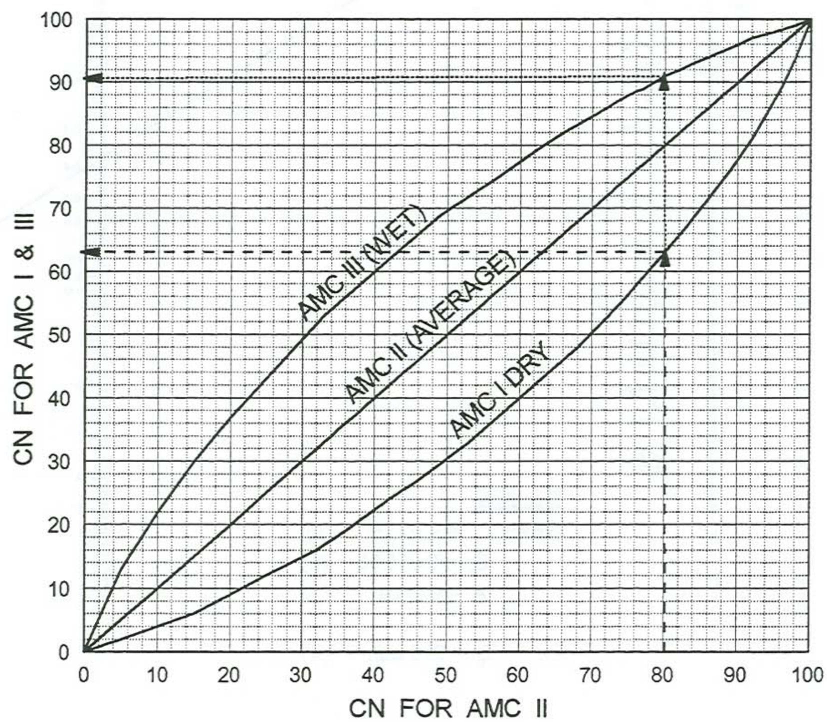
(i) All values are based on AMC II except those marked by * (AMC III) or ** (mean of AMC II and AMC III).

(ii) Values in brackets are AMC II and are to be used only for special cases.

(iii) Table is not applicable to frozen soils or to periods in which snowmelt contributes to runoff.

Design Chart 1.10: Antecedent Moisture Condition

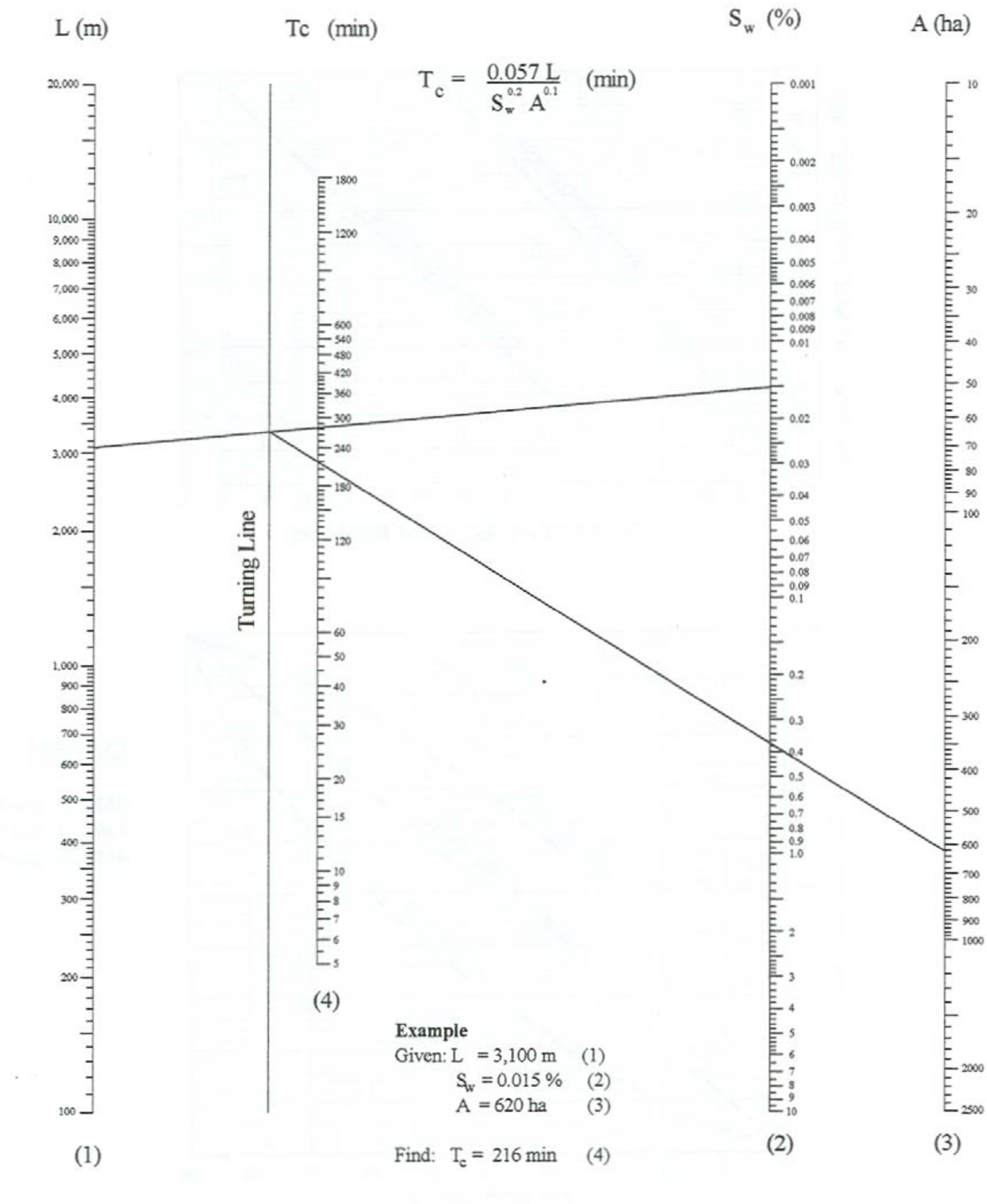




EXAMPLE

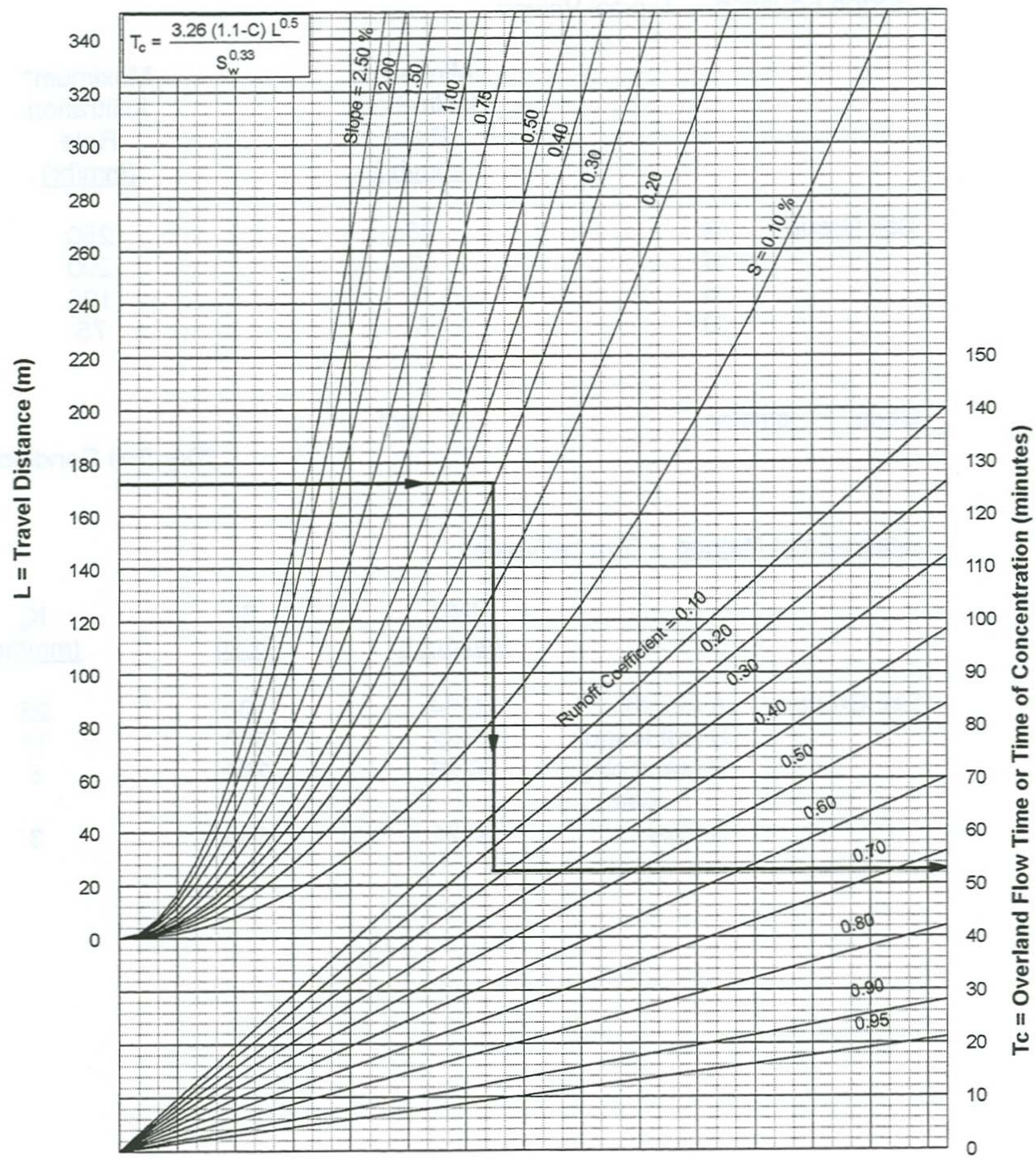
AMC II CN = 80
AMC I CN = 63
AMC III CN = 91

Design Chart 1.11: Time of Concentration - Bransby Williams Method



Source: French R., et al (1974)

Design Chart 1.12: Time of Concentration - Airport Method



Source: U.S. Department of Transportation (1970)

Design Chart 1.13: Infiltration Parameters

Horton Equation - Typical Values

		Minimum Infiltration Rate (mm/hr)	Maximum* Infiltration Rate (mm/hr)
Soil Group	A	25	250
	B	13	200
	C	5	125
	D	5	75

Decay Parameter 2 hr^{-1}

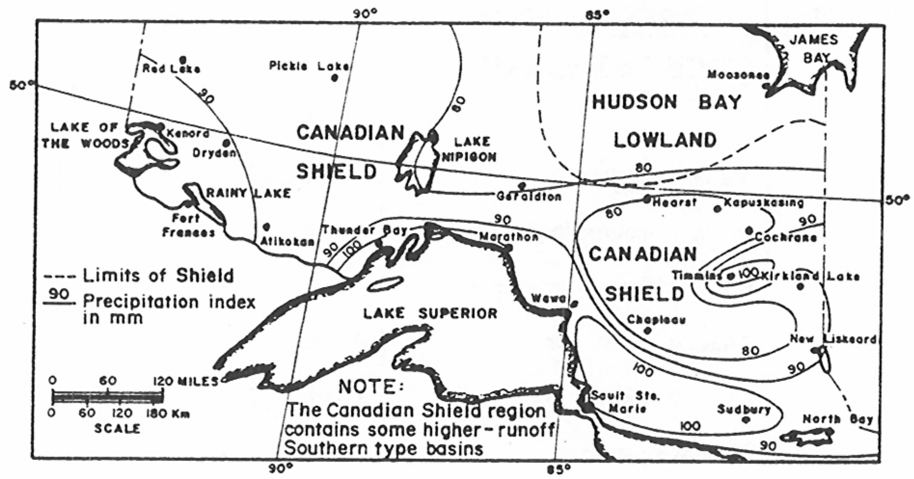
*Dry Soil Conditions

Green-Ampt Method - Typical Values

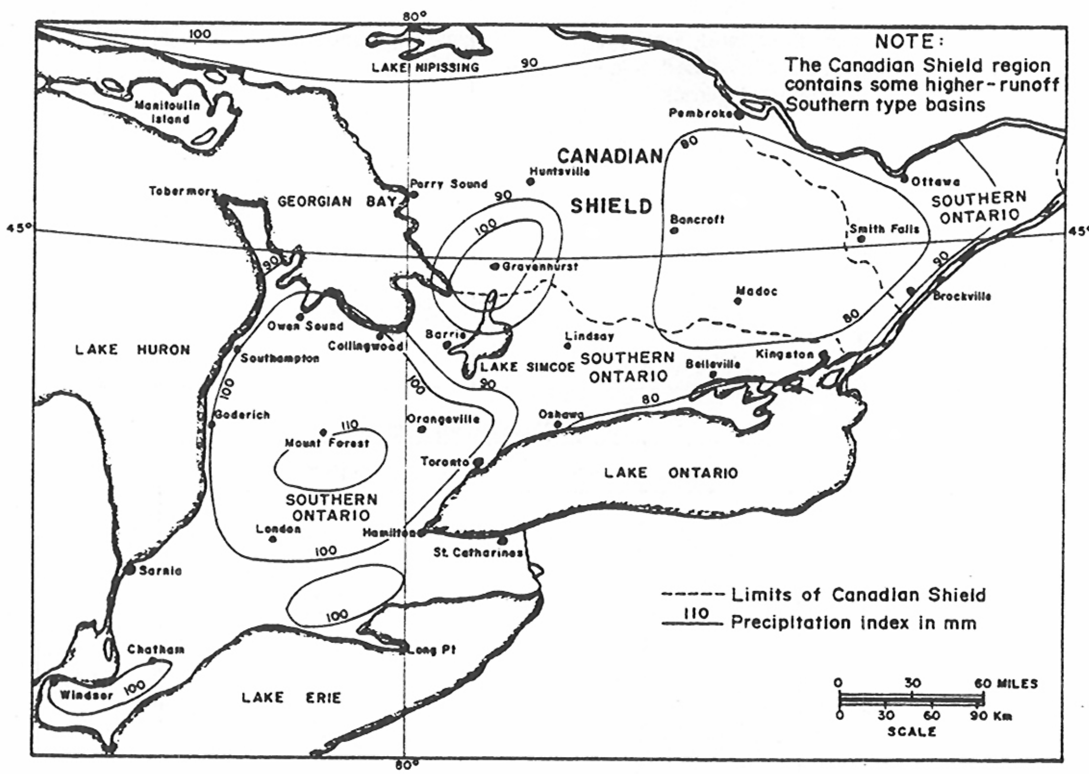
		IMD (mm/mm)	S_u (mm)	K_s (mm/hr)
Soil Group	A (sand)	0.34	100	25
	B (silt loam)	0.32	300	13
	C (sand clay loam)	0.26	250	5
	D (clay)	0.21	180	3

Source: M.L. Terstriep and J.B. Stall (1974)
U.S. EPA (1989)

Design Chart 1.14: Hydrologic Regions and Precipitation Index



(a) Northern Ontario



(b) Southern Ontario

Source: MTO (1985)

Design Chart 1.15: Typical Watershed Classes

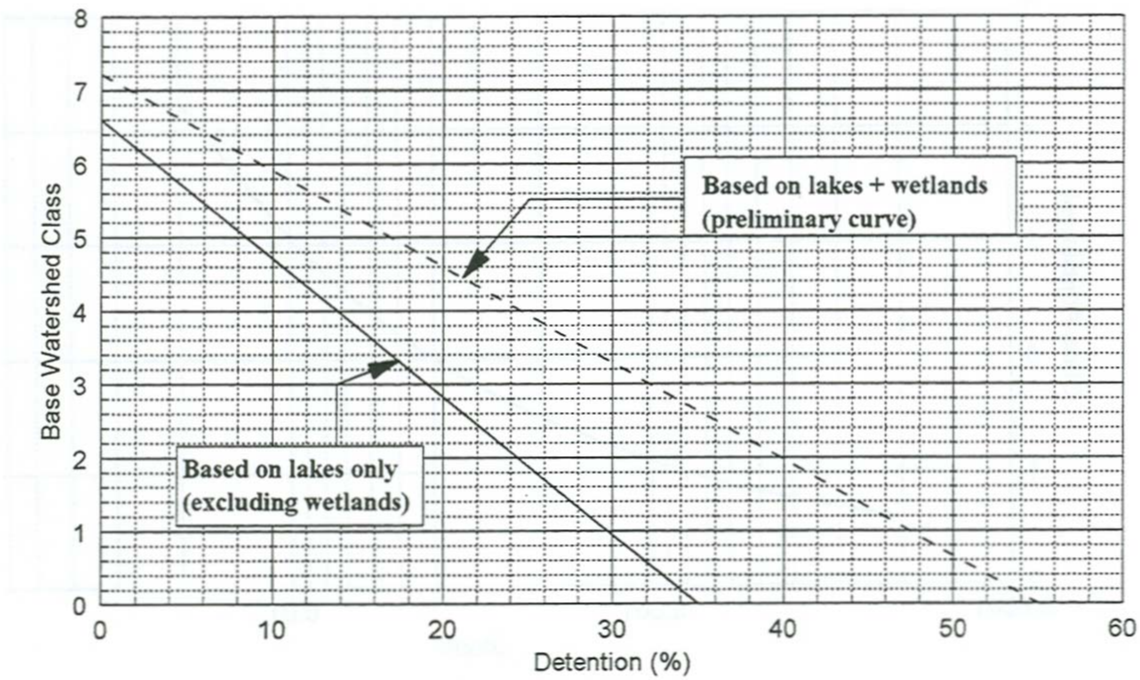
W/Shed Class	Predominant Soil Type	Land Use	Storage %
	SOUTHERN TYPE BASIN		
10	Clay Loam	Crop and pasture with some woodlots	Neg.
9	Medium textured loam	As class 10	"
8	Medium textured loam	Mostly wooded	"
	Medium loam on limestone	As class 10	"
	Shallow sandy loam	As class 10	"
7	Open sand soil	As class 10	"
	Shallow sandy loam	Mostly wooded	"
6	Deep sand or sand loam	" "	"
			% lakes
	SHIELD TYPE		
6	Shallow sandy loam on Precambrian bedrock, with some exposed bedrock.	Mostly wooded	3%
5	"	" "	8%
4	"	" "	14%
3	"	" "	19%
2	"	" "	25%
1	"	" "	30%

Class Coefficient, C

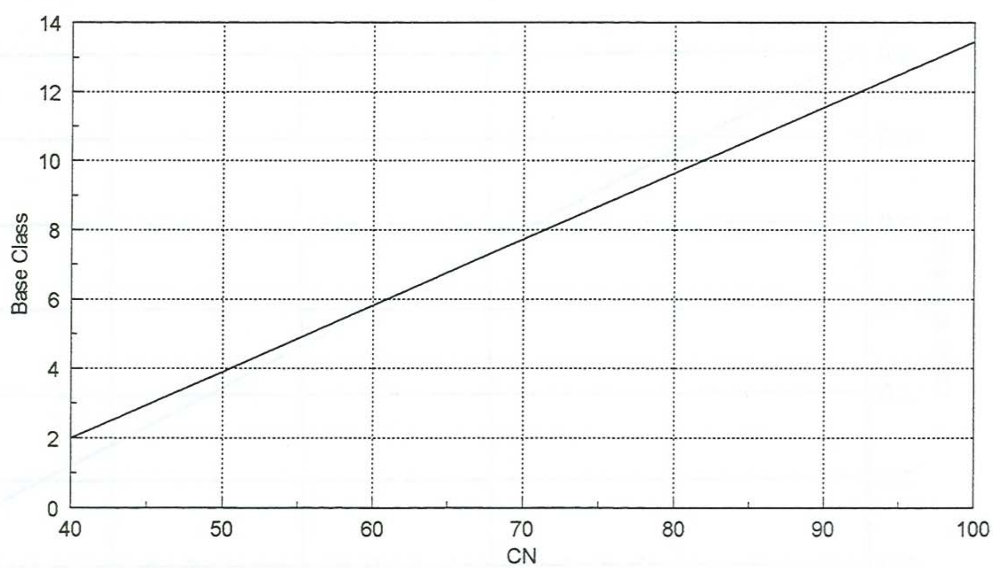
Watershed Class	Coefficient, C
1	0.15
2	0.22
3	0.31
4	0.44
5	0.63
6	0.90
7	1.29
8	1.84
9	2.62
10	3.74
11	5.34
12	7.63

Source: Whitely, et al (1995)

Design Chart 1.16: Base Class Chart Determination: Northern Basins



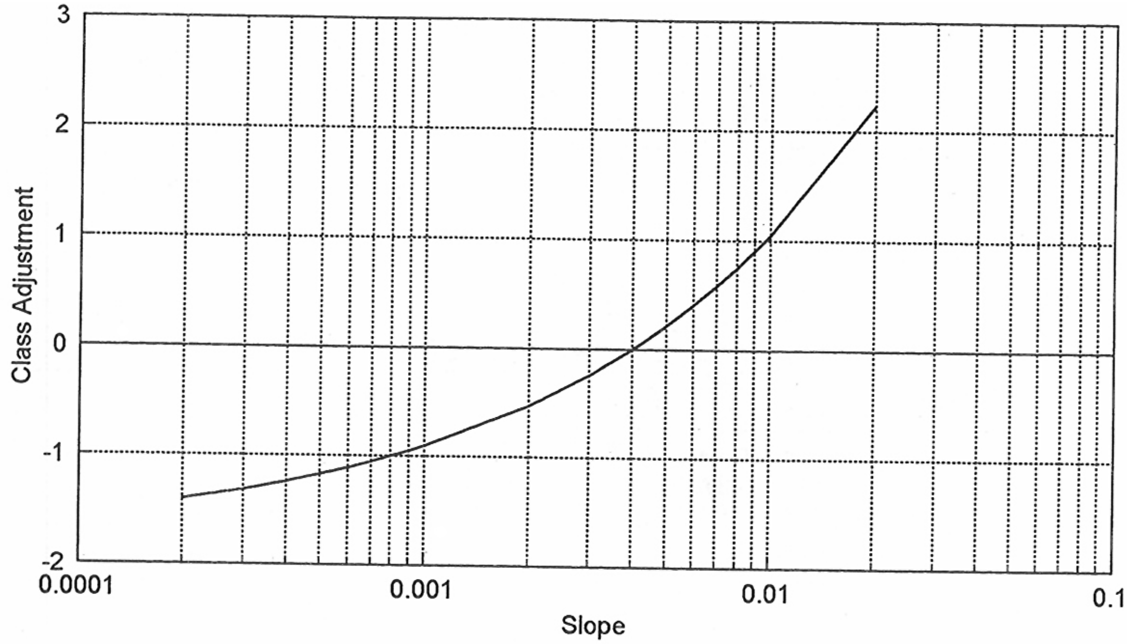
Design Chart 1.17: Base Class Determination - Southern Watersheds



Relationship Between Base Class and CN

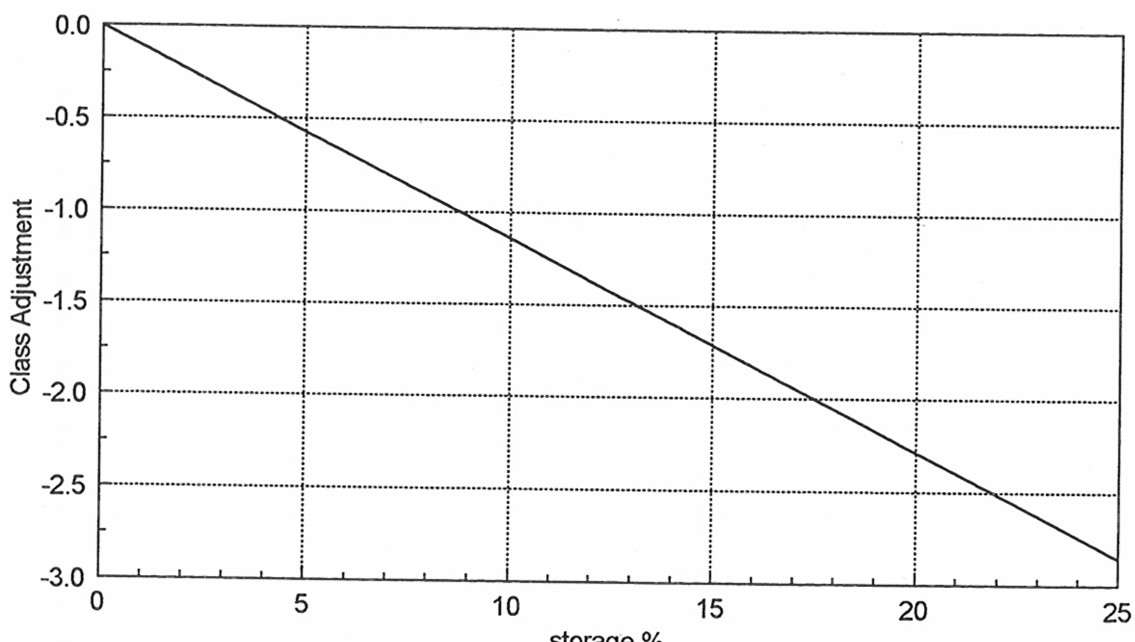
Source: Whitely, et al (1995)

Design Chart 1.18: Base Class Adjustment for Slope - Southern Basin



Relationship Between Class Adjustment and Slope

Design Chart 1.19: Base Class Adjustment for Detention - Southern Basins



Class Adjustment Due to Storage

Source: Whitely, et al (1995)

Design Chart 1.20: Regional Regression Factors - Northern Ontario Method

T (years)	2	2.33	5	10	25	50	100
CS	Value of K_T						
0.5	-0.09	0.09	0.81	1.34	1.93	2.32	2.67
0.6	-0.10	0.08	0.80	1.34	1.95	2.36	2.74
0.7	-0.11	0.07	0.79	1.34	1.97	2.40	2.80
0.8	-0.12	0.05	0.78	1.33	1.98	2.43	2.86
0.9	-0.13	0.05	0.77	1.33	1.99	2.46	2.90
1.0	-0.13	0.04	0.77	1.33	2.00	2.48	2.93
1.1	-0.16	0.00	0.72	1.30	2.04	2.59	3.14
1.2	-0.16	0.00	0.72	1.30	2.04	2.59	3.14
1.3	-0.18	-0.01	0.70	1.29	2.05	2.63	3.21
1.4	-0.18	-0.02	0.68	1.28	2.06	2.66	3.27
1.5	-0.19	-0.03	0.67	1.27	2.06	2.68	3.31
1.6	-0.20	-0.04	0.66	1.26	2.07	2.70	3.35
1.7	-0.20	-0.05	0.65	1.25	2.07	2.71	3.39
1.8	-0.21	-0.05	0.64	1.24	2.07	2.73	3.42
1.9	-0.21	-0.06	0.63	1.23	2.07	2.74	3.45
2.0	-0.21	-0.07	0.62	1.22	2.06	2.75	3.48
2.1	-0.22	-0.07	0.61	1.21	2.06	2.76	3.50
2.2	-0.22	-0.08	0.60	1.20	2.06	2.76	3.53
2.3	-0.22	-0.08	0.59	1.20	2.06	2.77	3.55
2.4	-0.22	-0.08	0.58	1.19	2.05	2.77	3.56
2.5	-0.23	-0.09	0.57	1.18	2.05	2.78	3.58
2.6	-0.23	-0.09	0.56	1.17	2.04	2.78	3.59
2.7	-0.23	-0.09	0.56	1.16	2.04	2.78	3.61
2.8	-0.23	-0.10	0.55	1.15	2.03	2.78	3.62
2.9	-0.23	-0.10	0.54	1.15	2.03	2.79	3.63
3.0	-0.24	-0.10	0.54	1.14	2.03	2.79	3.64
3.1	-0.24	-0.10	0.53	1.13	2.02	2.79	3.65

Source: Watt (1994)

Design Chart 2.01: Manning Roughness Coefficient

	Manning Roughness Coefficients
I. Sewers	
A. Concrete pipe storm sewers	0.011 - 0.013
B. Verified clay pipe	0.012 - 0.014
C. Steel pipe (smooth)	0.009 - 0.011
D. Monolithic concrete:	
1. Wood forms, rough	0.015 - 0.017
2. Wood forms, smooth	0.012 - 0.014
3. Steel forms	0.012 - 0.013
E. Cemented rubble masonry walls:	
1. Concrete floor and top	0.017 - 0.022
2. Natural floor	0.019 - 0.025
F. Laminated treated wood	0.015 - 0.017
G. Smooth walled polyethylene pipe	0.011 - 0.013
Corrugated interior polyethylene pipe (tentative)	0.024
H. Corrugated steel pipe or pipe arch	
68 x 13 mm corrugation (riveted, annular)	
Unpaved	0.024
25% paved	0.021
100% paved	0.012
68 x 13 mm helical	
Unpaved: 600 to 1525 mm ϕ range:	0.016 - 0.024
25% paved: 600 to 1525 mm ϕ range:	0.015 - 0.021
100% paved: all sizes	0.012
68 x 25 mm riveted (annular)	
Unpaved	0.027
25% paved	0.023
100% paved	0.012
76 x 25 mm helical	
Unpaved: 900 to 1980 mm dia.:	0.021 - 0.027
25% paved: 900 to 1980 mm dia.:	0.019 - 0.023
100% paved: all sizes	0.012
152 x 51 mm corrugation (annular)	
Unpaved 1550 - 4500 mm dia.or	0.030 - 0.033
1900 to 5050 mm span	0.026
25% paved	
II. Road Gutters	0.012
A. Concrete gutter, trowelled finish	
B. Asphalt pavement:	0.013
1. Smooth texture	0.016
2. Rough texture	
C. Concrete gutter with asphalt pavement:	
1. Smooth	0.013
2. Rough	0.015

Design Chart 2.01 (Continued)

	Manning Roughness Coefficients
D. Concrete pavement:	
1. Float finish	0.014
2. Broom finish	0.016
E. Brick	0.016
For gutters with small slope where sediment may accumulate, increase values by 0.002.	
III. Lined Open Channels	
A. Concrete, with surfaces as indicated:	
1. Formed, no finish	0.013 - 0.017
2. Trowel finish	0.012 - 0.014
3. Float finish	0.013 - 0.015
4. Float finish, some gravel on bottom	0.015 - 0.017
5. Gunite, good section	0.016 - 0.019
6. Gunite, wavy section	0.018 - 0.022
B. Concrete bottom float-finished, sides as indicated:	
1. Dressed stone in mortar	0.015 - 0.017
2. Random stone in mortar	0.017 - 0.020
3. Cement rubble masonry	0.020 - 0.030
4. Dry rubble (riprap)	0.020 - 0.030
C. Gravel bottom, sides as indicated:	
1. Formed concrete	0.017 - 0.020
2. Random stone mortar	0.020 - 0.023
3. Dry rubble (riprap)	0.023 - 0.033
D. Asphalt	
1. Smooth	0.013
2. Rough	0.016
E. Wood, planed, clean	0.011 - 0.013
F. 1. Good section	0.017 - 0.020
2. Irregular section	0.022 - 0.027
G. Riprap	0.035 - 0.040
H. Rock cut	0.025 - 0.045
IV. Unlined Open Channels	
A. Earth, uniform section:	
1. Clean, recently completed	0.016 - 0.018
2. Clean, after weathering	0.018 - 0.020
3. With short grass, few weeds	0.022 - 0.027
4. In gravelly, soil, uniform section, clean	0.022 - 0.025
B. Earth, fairly uniform section:	
1. No vegetation	0.022 - 0.025
2. Grass, some weeds	0.030 - 0.035
3. Dense weeds in deep channels	0.030 - 0.035
4. Sides clean, gravel bottom	0.025 - 0.030
5. Sides clean, cobble bottom	0.030 - 0.040

Design Chart 2.01 (Continued)

	Manning Roughness Coefficients
C. Dragline excavated or dredged:	
1. No vegetation	0.028 -0.033
2. Light brush on banks	0.035 -0.050
D. Rock:	
1. Based on design section	0.035
2. Based on actual mean section:	
a. Smooth and uniform	0.035 -0.040
b. Jagged and irregular	0.040 -0.045
E. Channels not maintained, vegetation uncut:	
1. Dense weeds, high as flow depth	0.08 - 0.12
2. Clean bottom, brush on sides	0.05 - 0.08
3. Clean bottom, brush on sides, high stage	0.07 - 0.11
4. Dense brush, high stage	0.10 - 0.14
V. Grassed Channels and Swales[2]	

Depth of Flow:	Up to 0.2 m	0.2 - 0.5 m
Velocity		
	0.6 m/s 1.8 m/s	0.6 m/s 1.8 m/s
A. Kentucky bluegrass:		
1. Mowed to 0.05 m		
2. Length 0.1 to 0.15 m	0.07 - 0.045	0.050 - 0.035
B. Good stand, any grass:	0.090 - 0.060	0.060 - 0.040
1. Length 0.30 m		
2. Length 0.60 m	0.180 - 0.090	0.120 - 0.070
C. Fair stand, any grass:	0.300 - 0.190	0.200 - 0.100
1. Length 0.30 m		
2. Length 0.60 m	0.140 - 0.080	0.100 - 0.060
	0.250 - 0.130	0.170 - 0.090

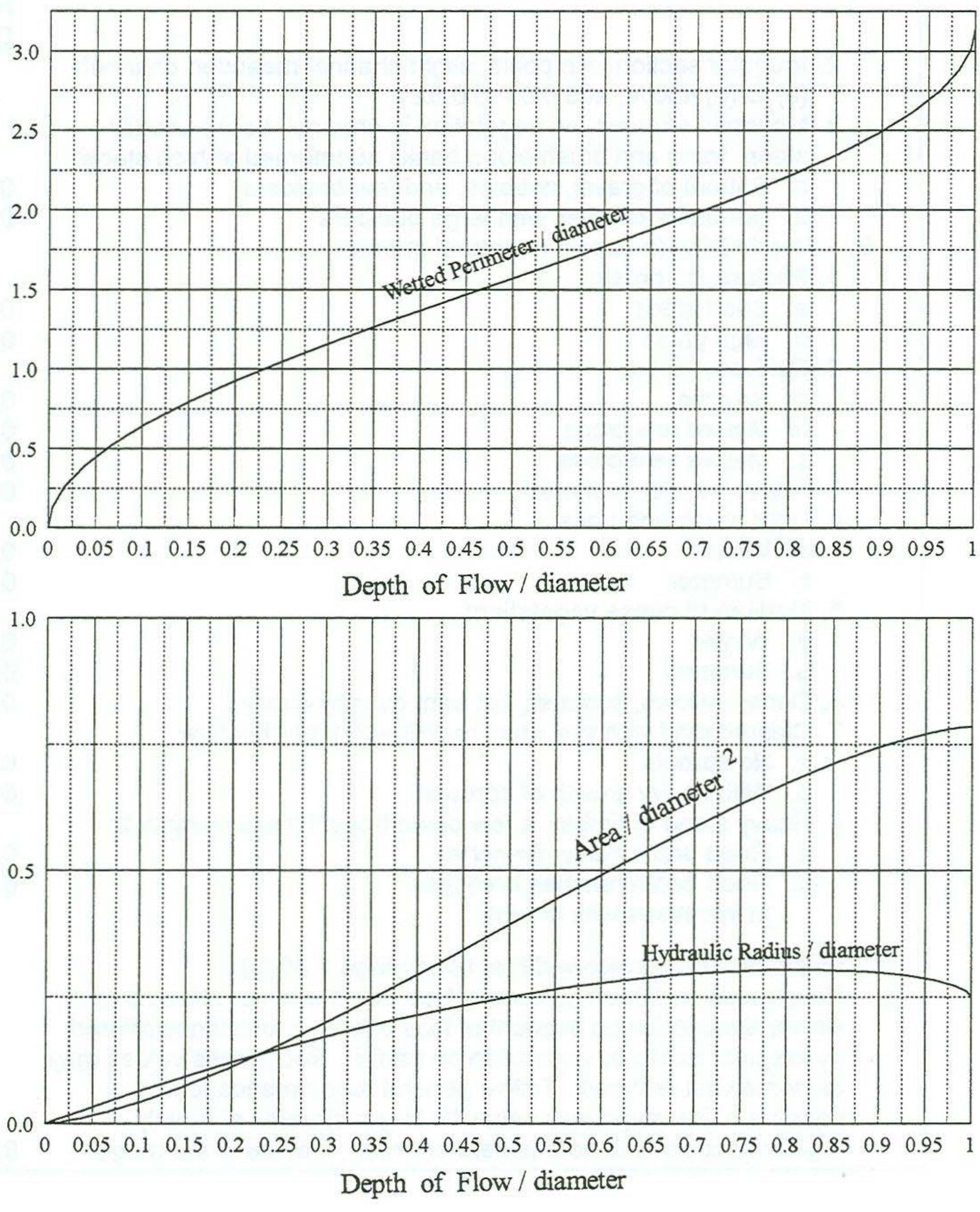
VI. Natural Watercourses	
A. Minor stream (surface width at flood stage < 30 m).	
1. Fairly regular section:	
a. Some grass and weeds, little or no brush	0.030 -0.035
b. Dense growth of weeds, depth of flow materially greater than weed height	0.035 -0.050
c. Some weeds, light brush on banks	0.035 -0.050
d. Some weeds, heavy brush on banks	0.050-0.070
e. Some weeds, dense willows on banks	0.060-0.080
f. For trees within channel with branches submerged at high stage, add 0.01 to 0.02 to above values.	

Design Chart 2.01 (Continued)

	Manning Roughness Coefficients
2. Irregular section with pools, slight channel meander; channels (a) to (e) above, add 0.01 to 0.02.	
3. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage:	
a. Bottom of gravel, cobbles, and few boulders	0.040 - 0.050
b. Bottom of cobbles with large boulders	0.050 - 0.070
B. Flood plains (adjacent to natural streams):	
1. Pasture, no brush:	
a. Short grass	0.030 - 0.035
b. High grass	0.035 - 0.050
2. Cultivated areas:	
a. No crop	0.030 - 0.040
b. Mature row crops	0.035 - 0.045
c. Mature field crops	0.040 - 0.050
3. Heavy weeds, scattered	0.050 - 0.070
4. Light brush and trees:	
a. Winter	0.050 - 0.060
b. Summer	0.060 - 0.080
5. Medium to dense vegetation:	
a. Winter	0.070 - 0.110
b. Summer	0.010 - 0.160
6. Dense willows, summer, not bent over by current	0.150 - 0.200
7. Cleared land with tree stumps, 250 - 370 per hectare	
a. No sprouts	0.040 - 0.050
b. With heavy growth of sprouts	0.060 - 0.080
8. Heavy stand of timber, a few down trees, little undergrowth:	
a. Flood depth below branches	0.100 - 0.120
b. Flood depth reaches branches (n increases with depth)	0.120 - 0.160
C. Major stream (surface width at flood stage > 30 m): Roughness coefficient is usually less than for minor streams of similar description on account of less effective resistance offered by irregular banks or vegetation on banks. Roughness values may be somewhat reduced. Follow general recommendations if possible. The roughness value for larger streams of mostly regular section, with no boulders or brush, may be in the range.	0.028 - 0.033

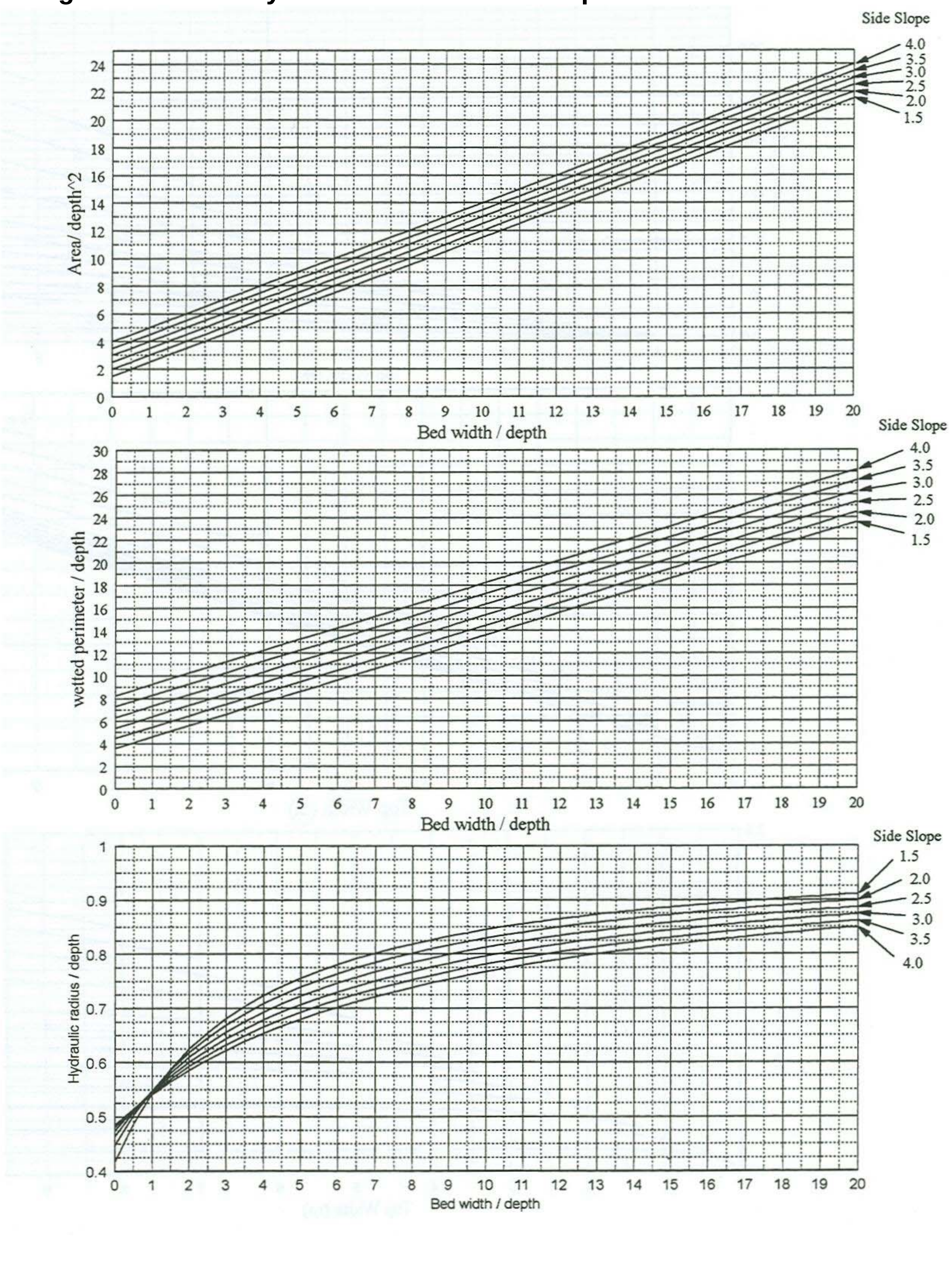
Sources: American Iron and Steel Institute (1980); Herr, L.A. et al, (1965)
Searcy, J.K. (1969)
Bradley, J.N. (1978)

Design Chart 2.02: Hydraulic Elements of Circular Pipes



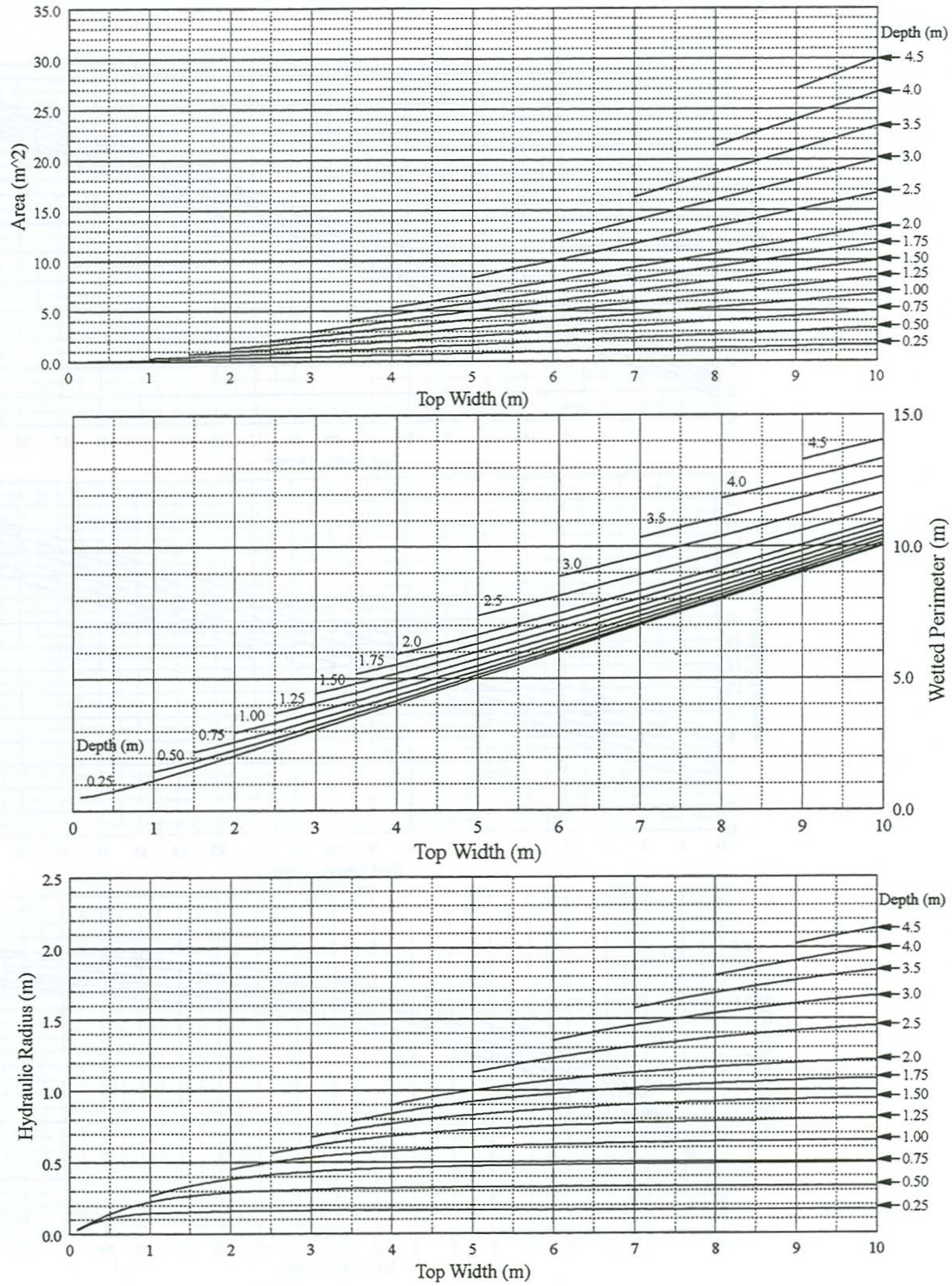
Source: Ministry of Transportation, MTO (1996)

Design Chart 2.03: Hydraulic Elements of Trapezoidal Channels



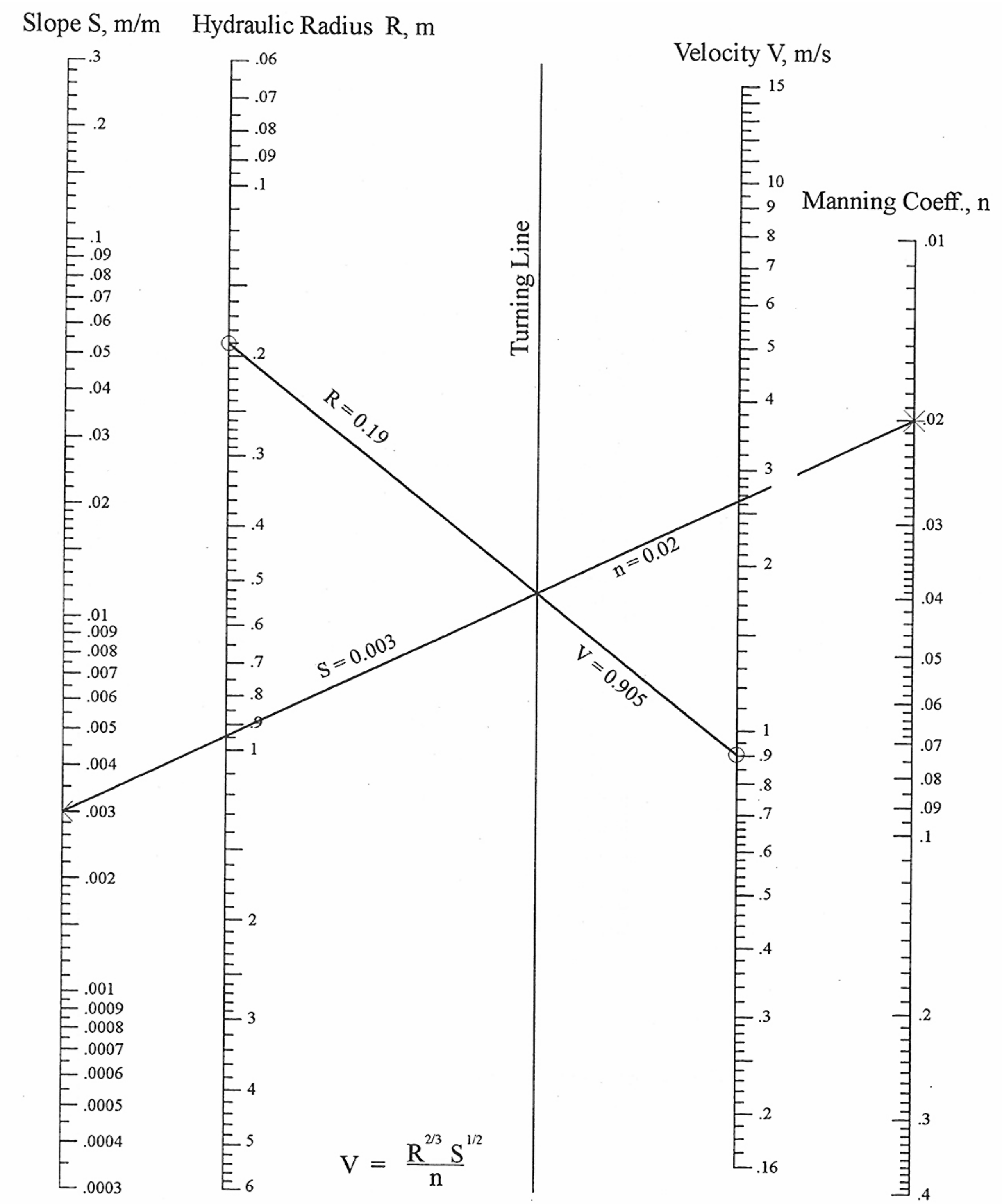
Source: Ministry of Transportation , MTO (1996)

Design Chart 2.04: Hydraulic Elements of Parabolic Channels



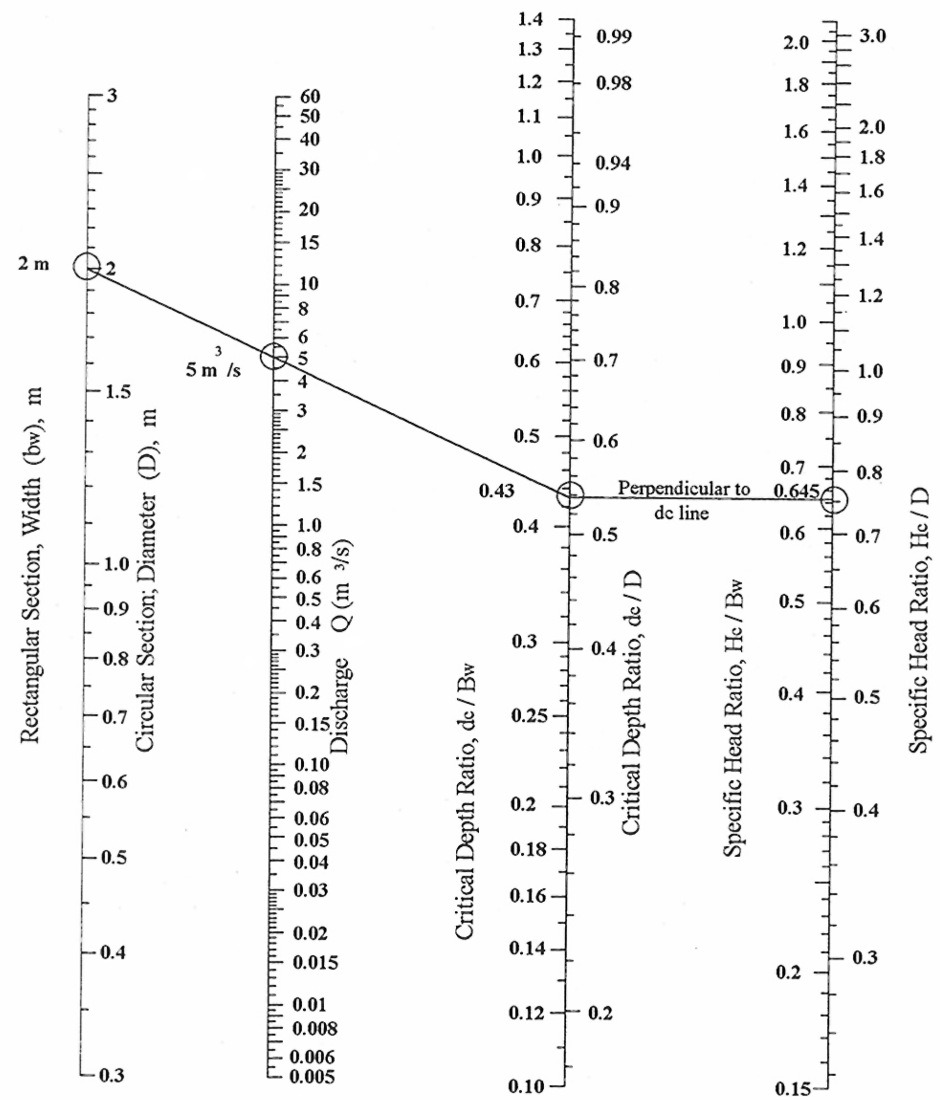
Source: Ministry of Transportation, MTO (1996)

Design Chart 2.05: Solving for Manning Equation



Source: Roads and Transportation Association of Canada (1982)

Design Chart 2.06: Solving for Critical Depth



(Note: Original converted to metric units.)

Source: American Society of Civil Engineers (1976)

Design Chart 2.07: Transition Loss of Coefficients: Bridges and Channels

SITUATION	K(ent)	K(ext)
Natural Reach (normal cross-section change)	0.1	0.3
Bridge (fills placed beyond normal channel width)	0.3	0.5
Guide Banks (each)	0.2	0.4
Groynes (each)	0.3	0.5
Dikes	0.1	0.2
Channel Bends Gradual Medium Severe	 0.1 0.2 0.3	 0.2 0.3 0.4
Flow Separation per obstruction (pier)	0.1	0.1

Source: U.S. Army Corps of Engineers (1991)

Design Chart 2.08: Transition Loss Coefficients: Culverts

TYPE OF BARREL AND INLET

Pipe, Concrete	**Ke**
Projecting from fill, socked end	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end or pipe	0.2
Square-edge	0.5
Rounded (radius = 1/ 12D)	0.2
Miltered to conform to fill slope	0.7
End-Section conforming to fill slope (standard precast)	0.5
Bevelled edges, 33.7° or 45° bevels	0.2
Side-tapered or slope-tapered inlets	0.2

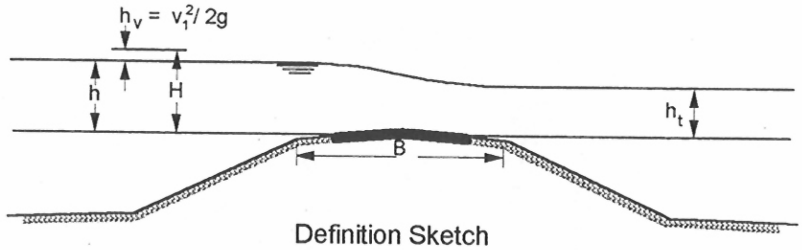
Pipe, or Pipe-Arch, Corrugated Steel	
Projecting from fill	0.9
Headwall or headwall and wingwalls, square edge	0.5
Mitered to conform to fill slope	0.7
End-Section conforming to fill slope (standard prefab)	0.5
Bevelled edges, 33.7° or 45° bevels	0.25
Side-tapered or slope-tapered inlets	0.2

Box, Reinforced Concrete	
Headwall	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius 1/12	
Barrel dimension, or bevelled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius 1/12	
barrel dimension, or bevelled top edge	0.2
Wingwalls at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parellel (extension of sides)	
Square edged at crown	0.7
Side-tapered or slope-tapered inlet	0.2
Projecting	
Square-edge	0.7*
Bevelled edges, 33.7° or 45° bevels	0.2*

* Estimated

Source: Harrison et al (1972), Herr et al (1977)

Design Chart 2.09: Solving for Weir Flow

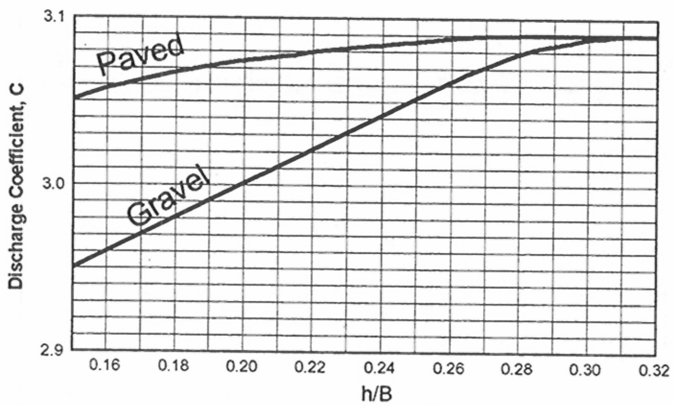


Definition Sketch

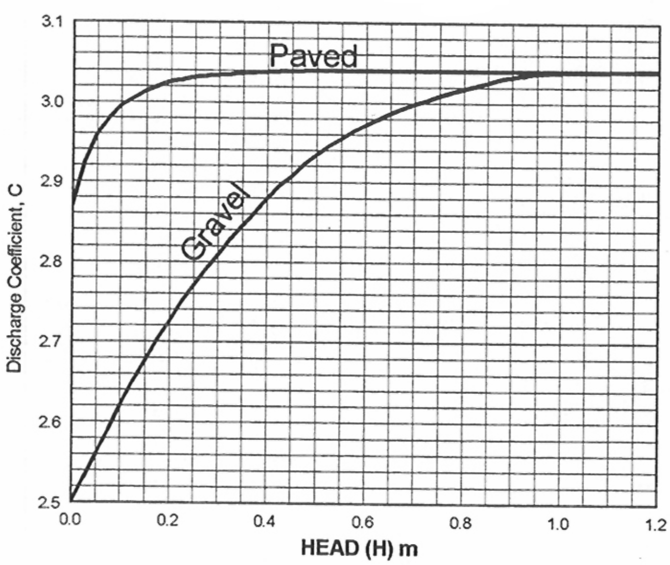
$$Q = \Sigma\, 0.55\, CLH^{1.5} k_t \; m^3/s$$

where ;

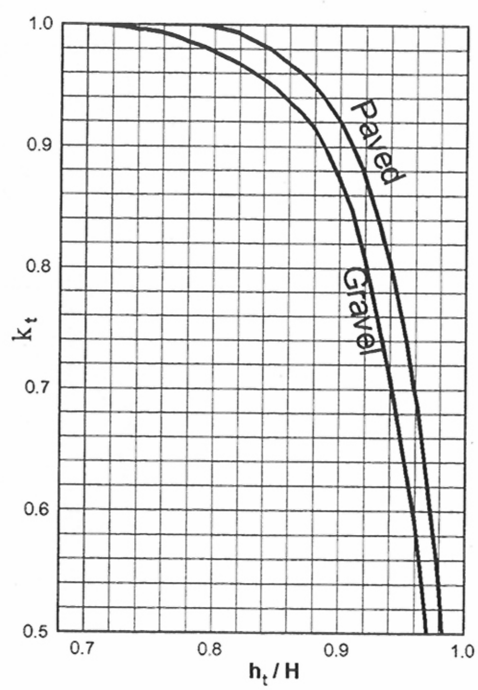
- Q = total overflow,
- C = discharge coefficient (given by chart (a) or (b)),
- L = length of overflow subsection along embankment, m,
- H = total head, m,
- k_t = submergence factor (given by chart (c))



(a) Discharge coefficient C for h / B > 0.15 (free flow)

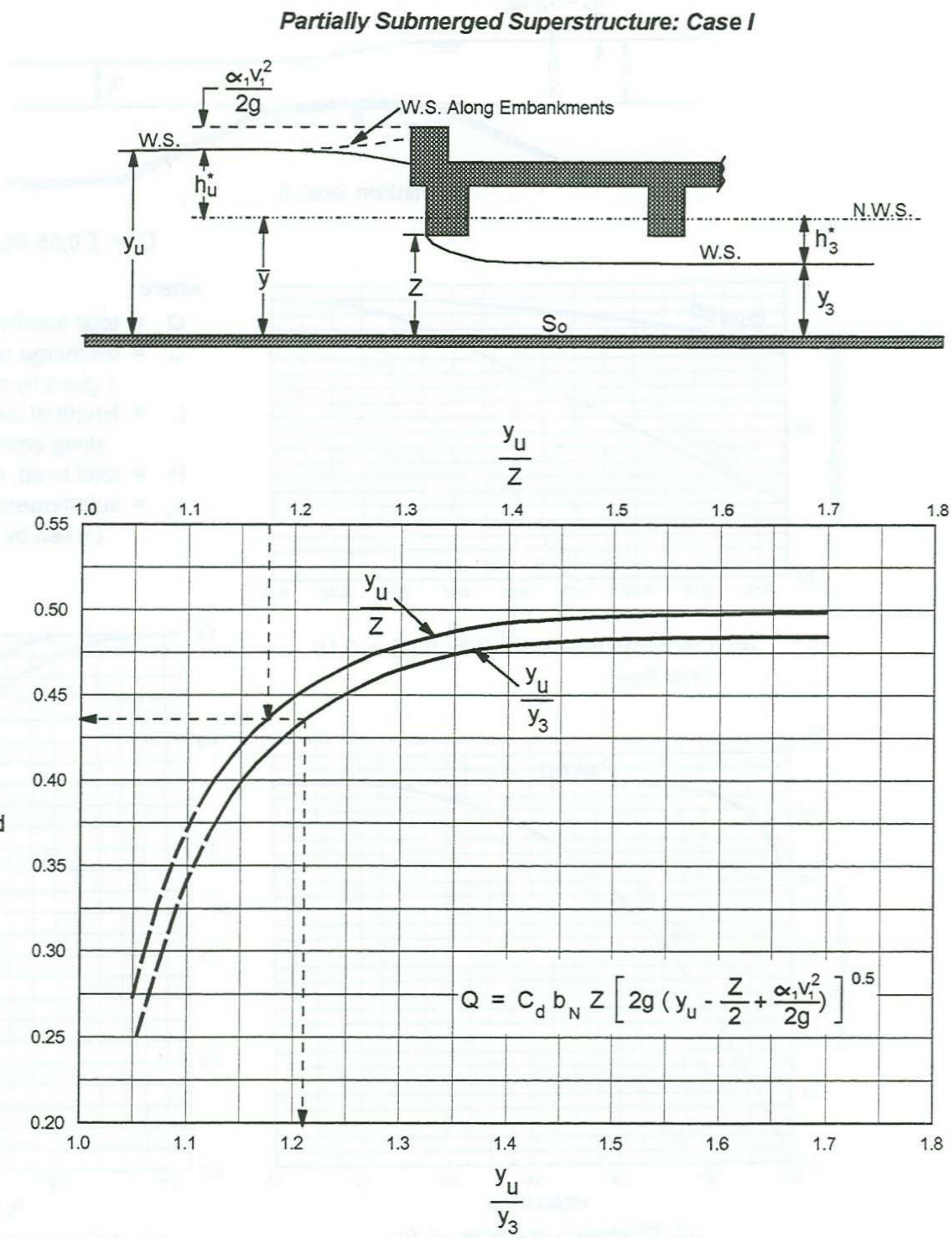


(b) Discharge coefficient C for h / B < 0.15 (free flow)



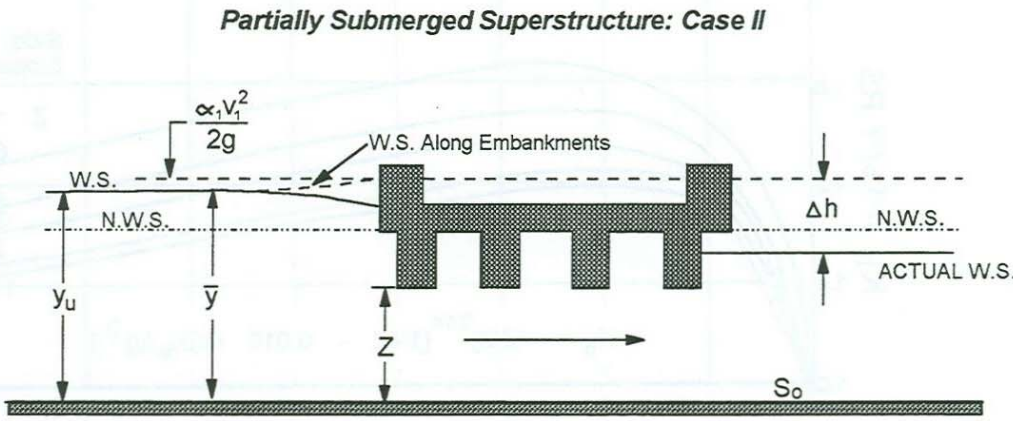
(c) Adjustment factor k_t for submerged flow

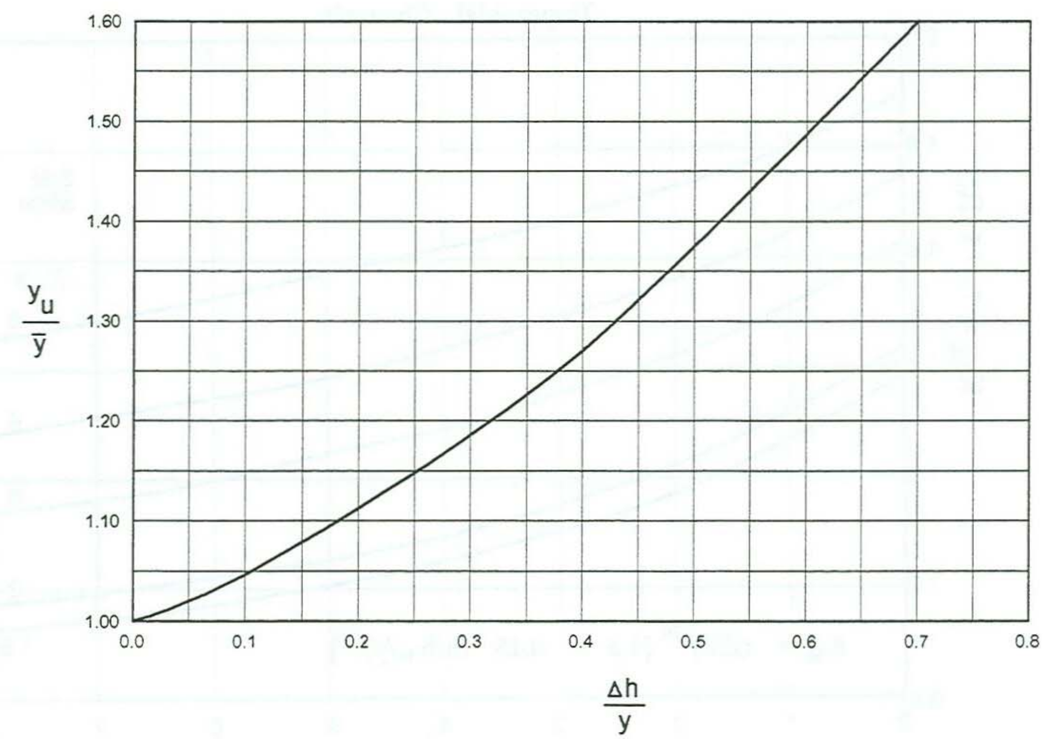
Design Chart 2.10: Solving for Pressure Flow



Source: Bradley (1973)

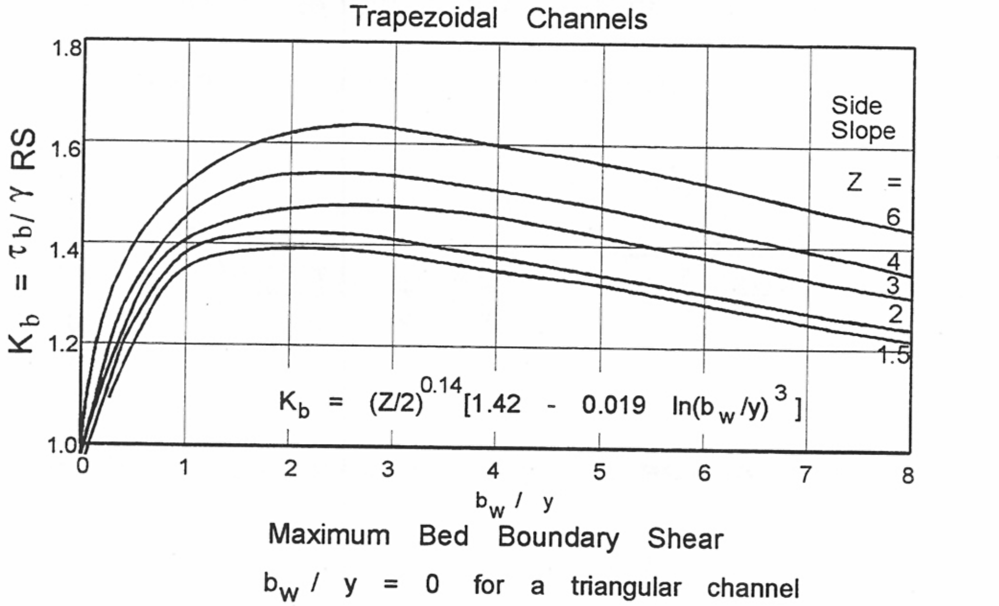
Design Chart 2.10 (continued): Solving for Pressure Flow



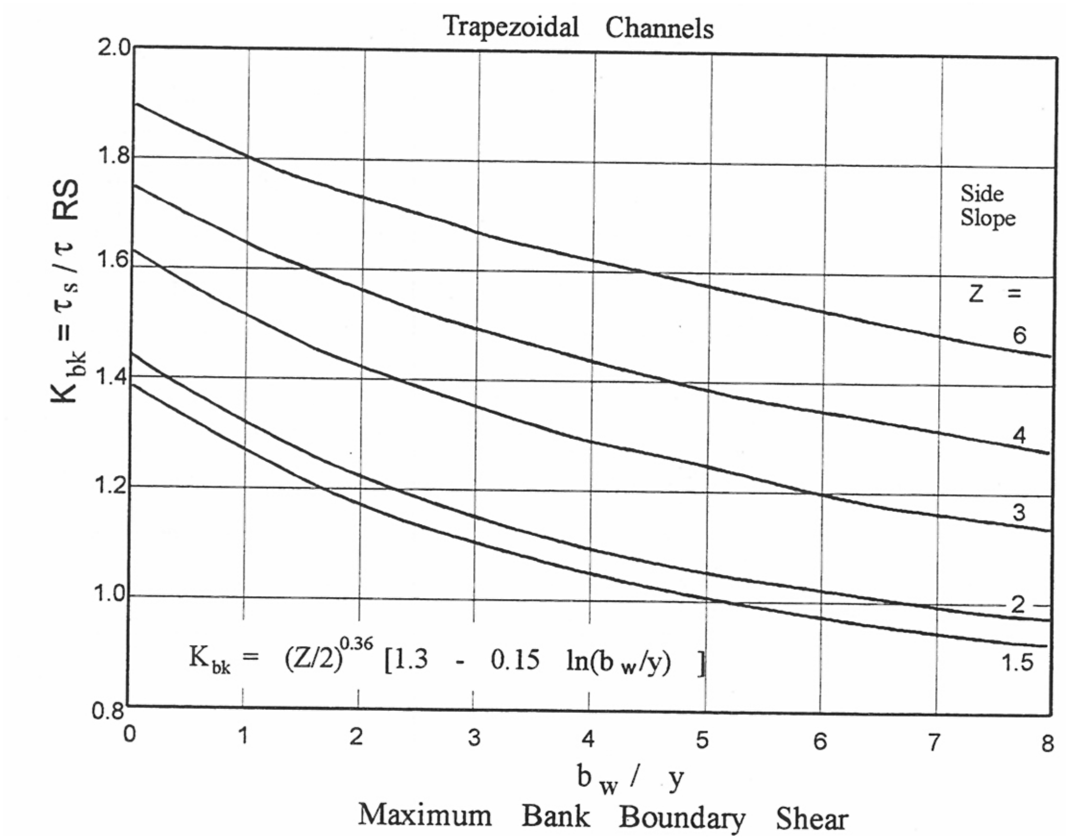


Source: Bradley (1973)

Design Chart 2.11: Coefficients of Boundary Shear on Channel Bed

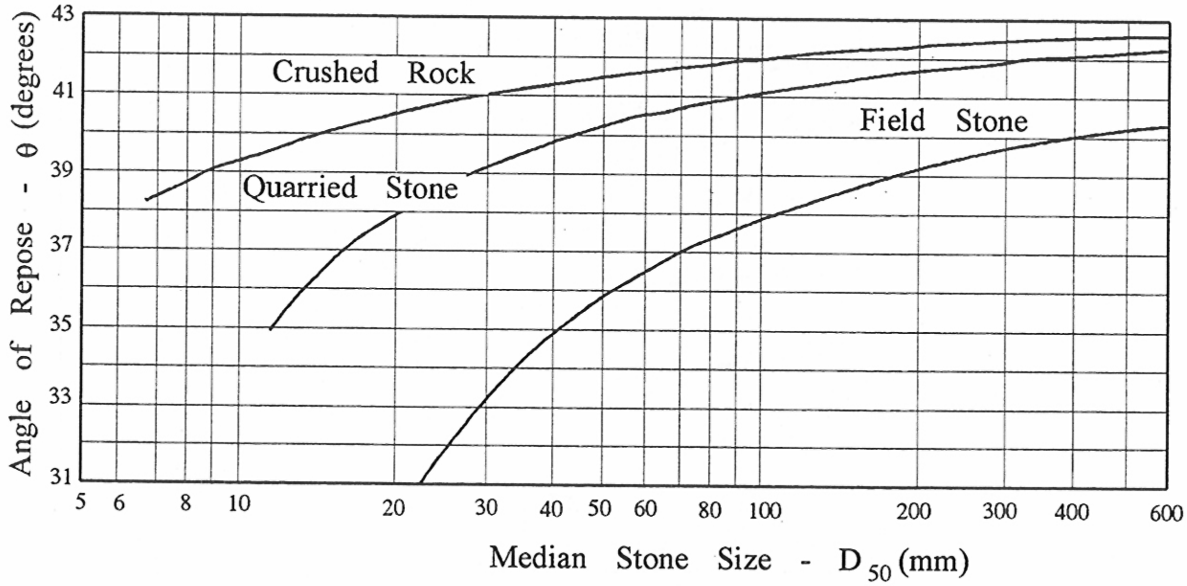


Design Chart 2.12: Coefficients of Boundary Shear on the Side Slope



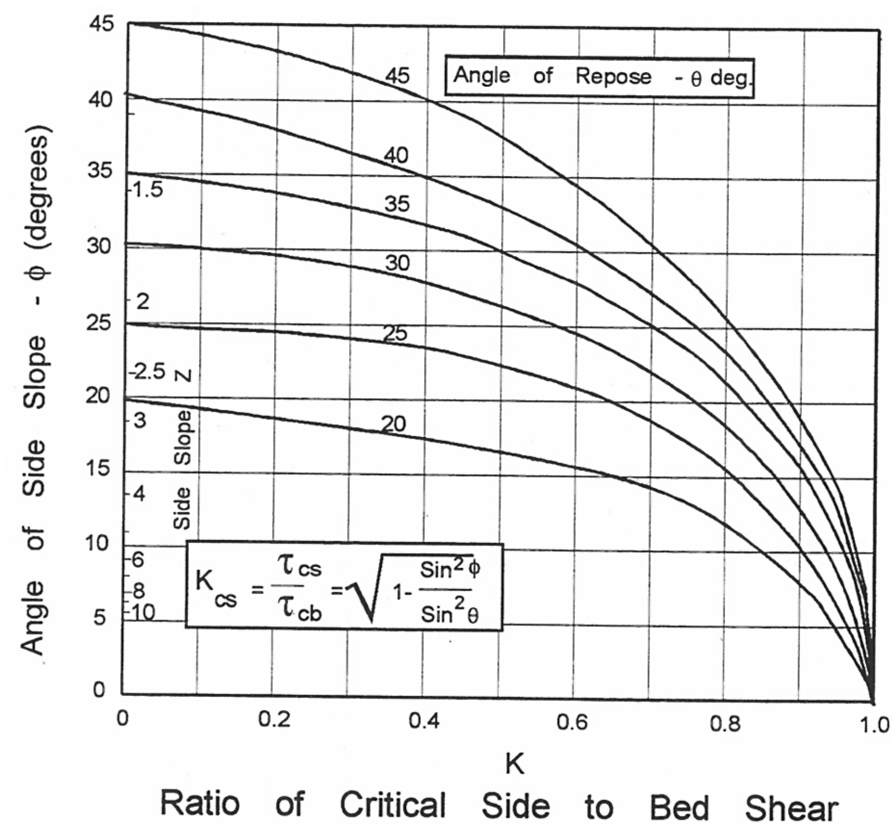
Source: After Tentative Design Procedures for Riprap Lined Channels, 1970, A.G. Anderson, A.S. Paintal and J.T. Davenport, National Cooperative Highway Research Program Report 108.

Design Chart 2.13: Determining Angle of Repose



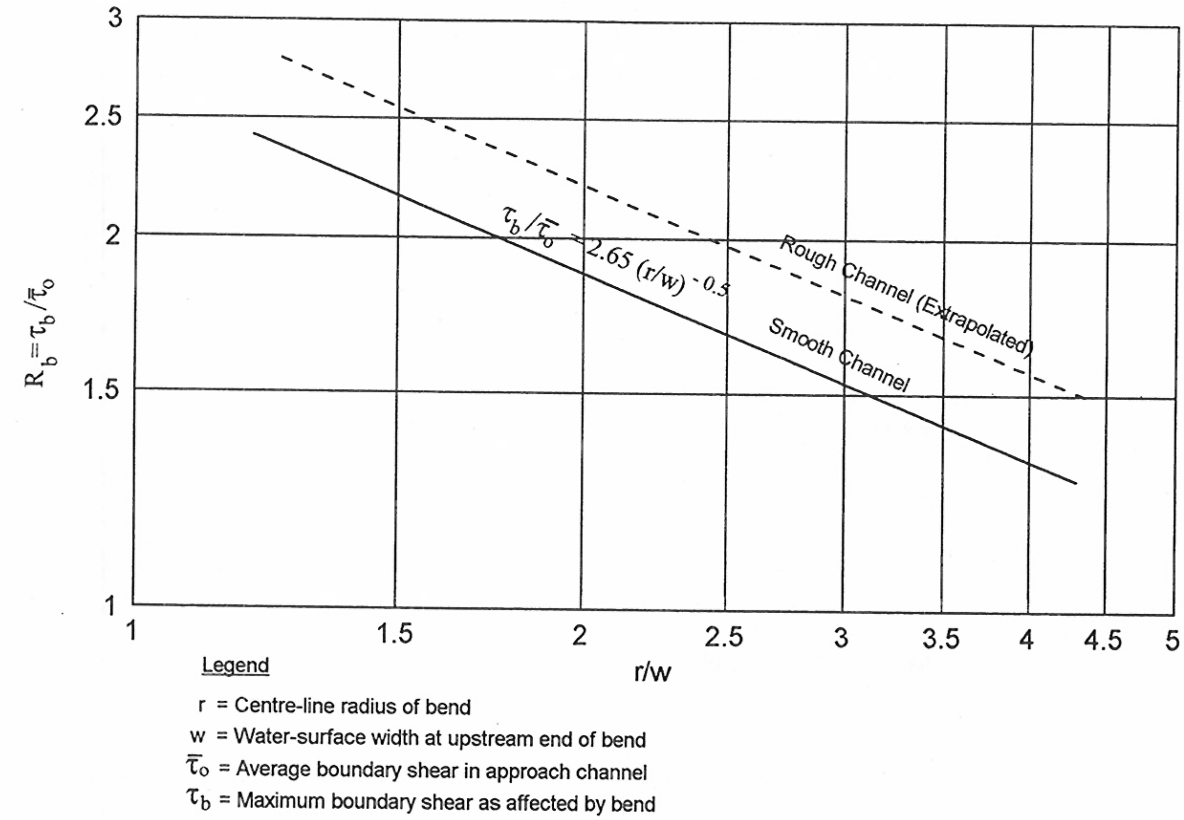
Source: After Tentative Design Procedures for Riprap Lined Channels, 1970, A.G. Anderson, A.S. Paintal and J.T. Davenport, National Cooperative Highway Research Program Report 108.

Design Chart 2.14: Coefficients of Resisting Shear on Side Slopes



Source: After Tentative Design Procedures for Riprap Lined Channels, 1970, A.G. Anderson, Paintal and J.T. Davenport, National Cooperative Highway Research Program Report 108.

Design Chart 2.15: Shear Coefficient for Outside of Channel Bends



Source: U.S. Army Corps of Engineers (1970)

Design Chart 2.16: Permissible Shear for Lining Materials

Vegetative			Permissible Unit Shear Stress (kg / m^2)	
Class A			18	
Class B			10	
Class C			4.9	
Class D			2.9	
Class E			1.7	
Gravel Riprap	1"	25 mm	1.6	Estimates only. Permissible shear stress is dependent on several factors including flow depth, velocity, bank side slope, etc.
	2"	50 mm	3.2	
Rock Riprap	6"	150 mm	9.8	
	12"	300 mm	20	

Note: Class A, B, C, D and E shown on Design Chart 2.23

Source: U.S. Department of Transportation (1988)

Design Chart 2.17: Maximum Permissive Flow Velocities - Native Material/Linings

	Velocity		
Material	Clear water (m/s)	Water carrying fine silts (m/s)	Water carrying sand and gravel (m/s)
Fine sand (noncolloidal)	0.45	0.75	0.50
Sandy loam (noncolloidal)	0.50	0.75	0.60
Silt loam (noncolloidal)	0.60	0.90	0.60
Ordinary firm loam	0.75	1.10	0.70
Volcanic ash	0.75	1.10	0.60
Fine gravel	0.75	1.50	1.15
Stiff clay (very colloidal)	1.15	1.50	0.90
Graded, loam to cobbles (noncolloidal)	1.15	1.50	0.50
Graded, silt to cobbles (colloidal)	1.20	1.70	1.50
Alluvial silts (noncolloidal)	0.60	1.10	1.60
Alluvial silts (collodial)	1.15	1.50	0.90
Coarse gravel (noncolloidal)	1.20	1.85	2.00
Cobbles and Shingles	1.50	1.70	2.00
Shales and hard plans	1.85	1.85	1.50

For sinuous channels multiply allowable velocity by 0.95 for slightly sinuous, by 0.9 for moderately sinuous channels, and by 0.8 for highly sinuous channels.
Source: American Society of Civil Engineers - ASCE (1926)

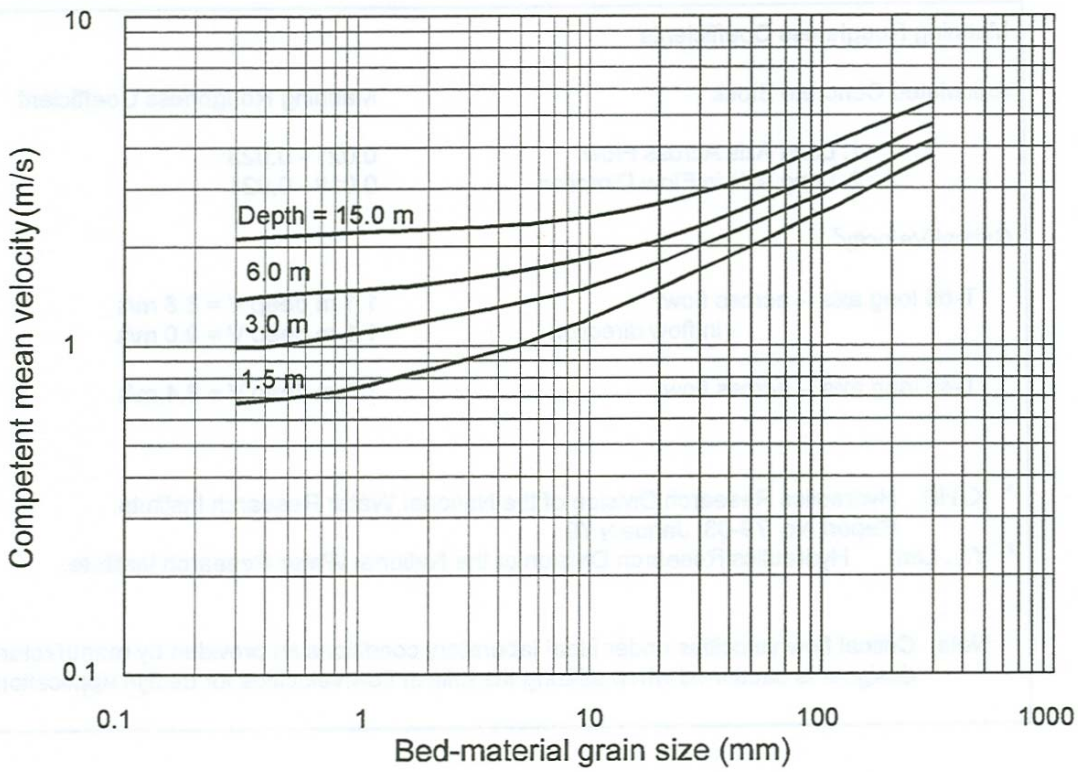
- Vegetal Linings

	Velocity		
Cover	Slope range (%)	Erosion resistant soils (m/s)	Easily eroded soils (m/s)
Bermuda grass	0-5	2.4	1.8
	5-10	2.1	1.5
	over 10	1.8	1.2
Buffalo grass	0-5	2.1	1.5
Kentucky Bluegrass	5-10	1.8	1.2
Smooth Brome	over 10	1.5	0.9
Grass mixture	0-5[3]	1.5	1.2
	1-10[3]	1.2	0.9
Lespedeza Sericea	0-5[4]	1.1	0.8
Common Lespedeza[5] Sudan grass[5]	0-5[4]	1.1	0.8

Use flow velocities over 1.5 m/s only where good cover and proper maintenance can be obtained.
Do not use on slopes steeper than 10 percent.
Use on slopes steeper than 5 percent is not recommended.
Annuals, used on mild slopes or as temporary protection until permanent covers are established.
Note: Permissible average flow velocities should be based on local experience whenever possible.

Source: U.S. Department of Agriculture (1954)

Design Chart 2.18: Permissible Velocity Chart: Cohensionless Soil



Source: Neill (1973)

Design Chart 2.19: Hydraulic Characteristics of Terrafix Blocks

Manning Roughness Coefficients

Articulated Concrete Block	Manning Roughness Coefficient
1. Long Axis Across Flow	0.021 - 0.023
2. Long Axis in Flow Direction	0.019 - 0.021

Critical Velocity[2]

T-60 long axis	- across flow	1.1 m deep V = 8.8 m/s
	- in flow direction	1.1 m deep V = 9.0 m/s
T-45 long axis	- across flow	1.5 m deep V = 8.4 m/s

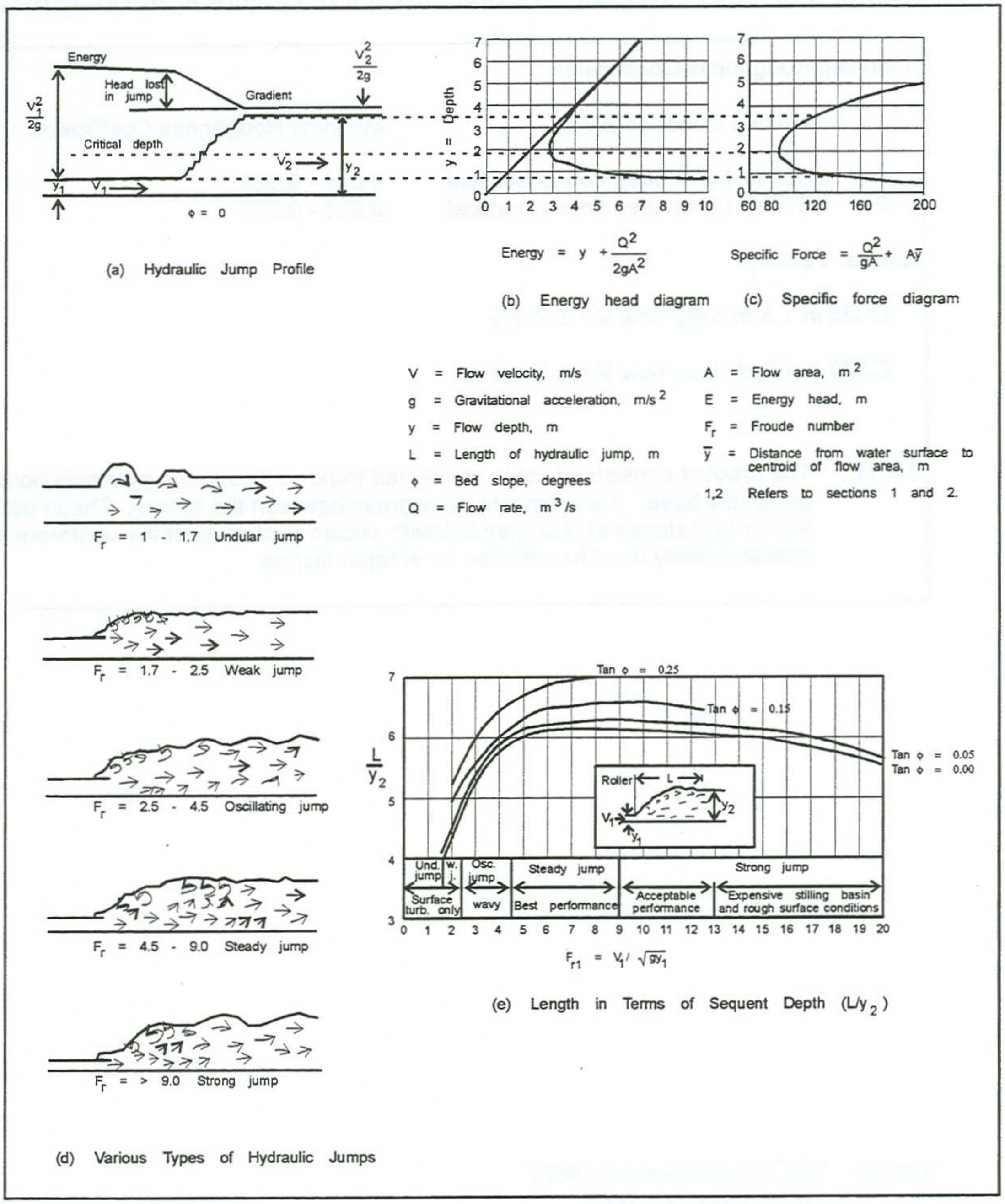
[1] K. Hill. Hydraulics Research Division of the National Water Research Institute. Report No. 79-03, January 79.

[2] Y.L. Lau. Hydraulics Research Division of the National Water Research Institute.

Note: Critical flow velocities under ideal laboratory conditions as provided by manufacturers. The designer is cautioned when utilizing the critical flow velocities for design applications.

Sources: Hill (1979)
Lau (1979)

Design Chart 2.20: Hydraulic Jump Length



Source: After Hydraulic Design of Stilling Basins and Energy Dissipators, 1974, A.J. Peterka.

Design Chart 2.21: Hydraulic Characteristics of Concrete Blocks with Cables

Manning Roughness Coefficients

Articulated Concrete Block	Manning Roughness Coefficient
1. CC35 in 1.5 m deep flow subcritical	0.024 - 0.026
2. CC70 in 1.5 m deep flow subcritical	0.028 - 0.032

Critical Velocity

CC35 in 1.5 m deep flow V = 5.7 m/s

CC70 in 1.5 m deep flow V = 8.0 m/s

NOTE: The product consists of cable connected truncated concrete pyramids bonded to a geotextile base. It is normal to have grass between the blocks. These design data are for bed slopes of less than 2% with secure anchoring of the upstream edge. The critical velocity must be reduced for steeper slopes.

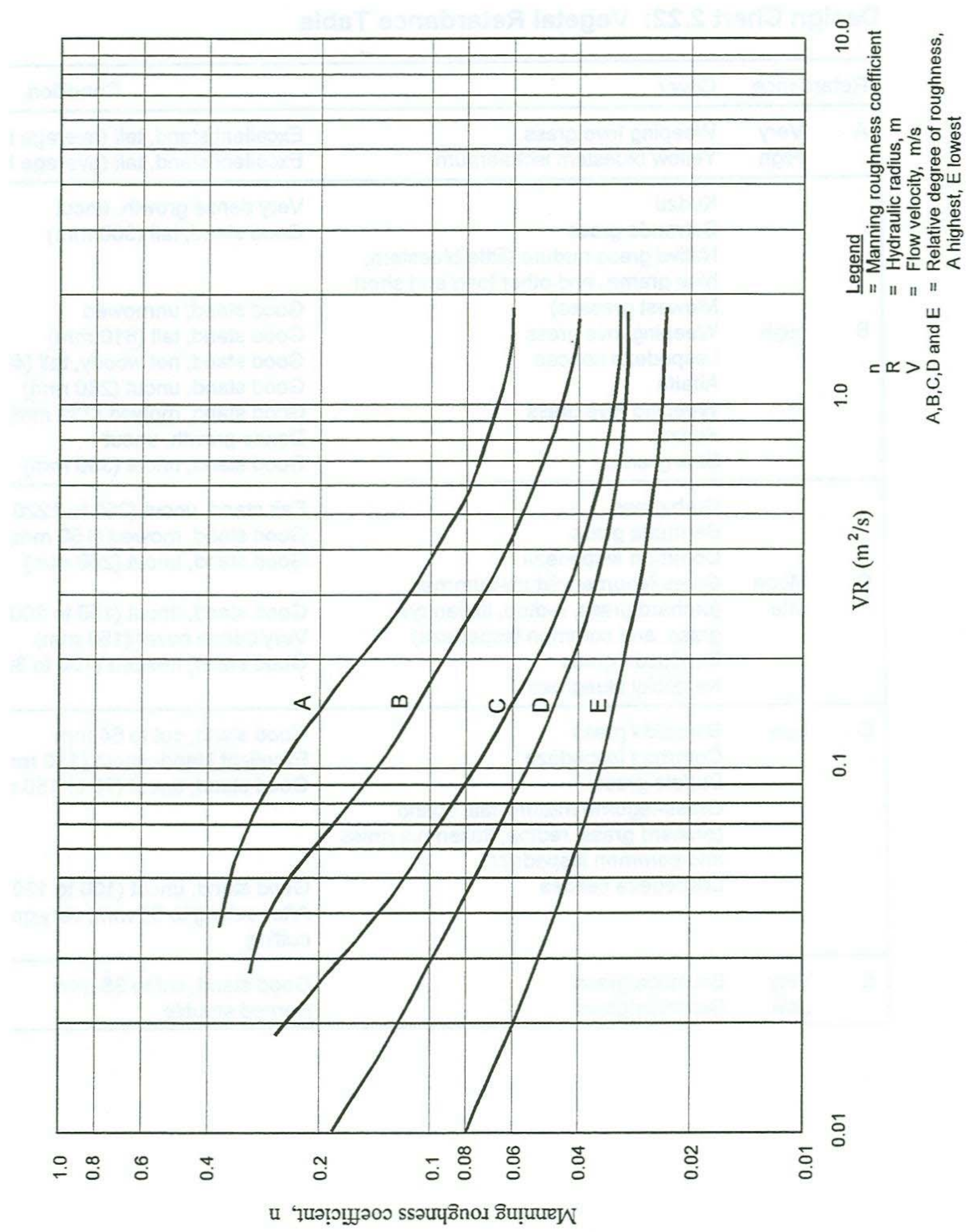
Source: McCorquodale et al (1988)
McCorquodale (1991)

Design Chart 2.22: Vegetal Retardance Table

Retardance	Cover	Condition
A Very High	Weeping love grass	Excellent stand, tall (average height 760 mm)
	Yellow bluestem ischaemum	Excellent stand, tall (average height 910 mm)
B High	Kudzu	Very dense growth, uncut
	Bermuda grass	Good stand, tall (300 mm)
	Native grass mixture (little bluestem, blue grama, and other long and short Midwest grasses)	Good stand, unmowed
	Weeping love grass	Good stand, tall (610 mm)
	Lespedeza sencea	Good stand, not woody, tall (480 mm)
	Alfalfa	Good stand, uncut (280 mm)
	Weeping love grass	Good stand, mowed (330 mm)
	Kudzu	Dense growth, uncut
	Blue grama	Good stand, uncut (330 mm)
C Moderate	Crab grass	Fair stand, uncut (250 to 1220 mm)
	Bermuda grass	Good stand, mowed (150 mm)
	Common lespedeza	Good stand, uncut (280 mm)
	Grass-legume mixture-summer (orchard grass, redtop, Italian rye grass, and common lespedeza)	Good stand, uncut (150 to 200 mm)
	Centipede grass	Very dense cover (150 mm)
	Kentucky bluegrass	Good stand, headed (150 to 300 mm)
D Low	Bermuda grass	Good stand, cut to 64 mm
	Common lespedeza	Excellent stand, uncut (110 mm)
	Buffalo grass	Good stand, uncut (76 to 150 mm)
	Grass-legume mixture-fall, spring (orchard grass, redtop, Italian rye grass and common lespedeza)	Good stand, uncut (100 to 130 mm)
	Lespedeza sericea	After cutting to 50 mm, very good stand before cutting
E Very Low	Bermuda grass	Good stand, cut to 38 mm
	Bermuda grass	Burned stubble

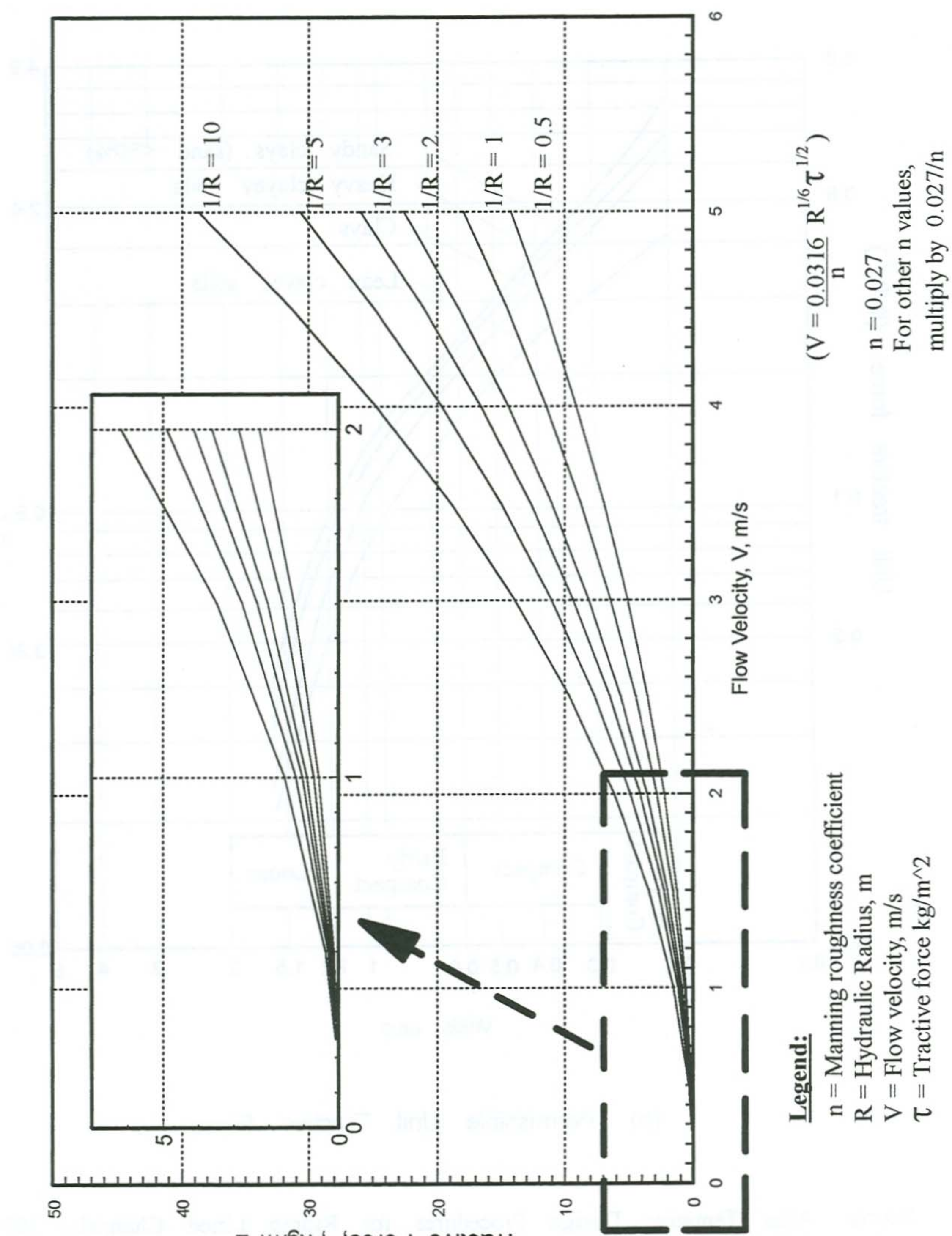
Source: U.S. Department of Agriculture (1954)

Design Chart 2.23: Vegetal Retardance Curves

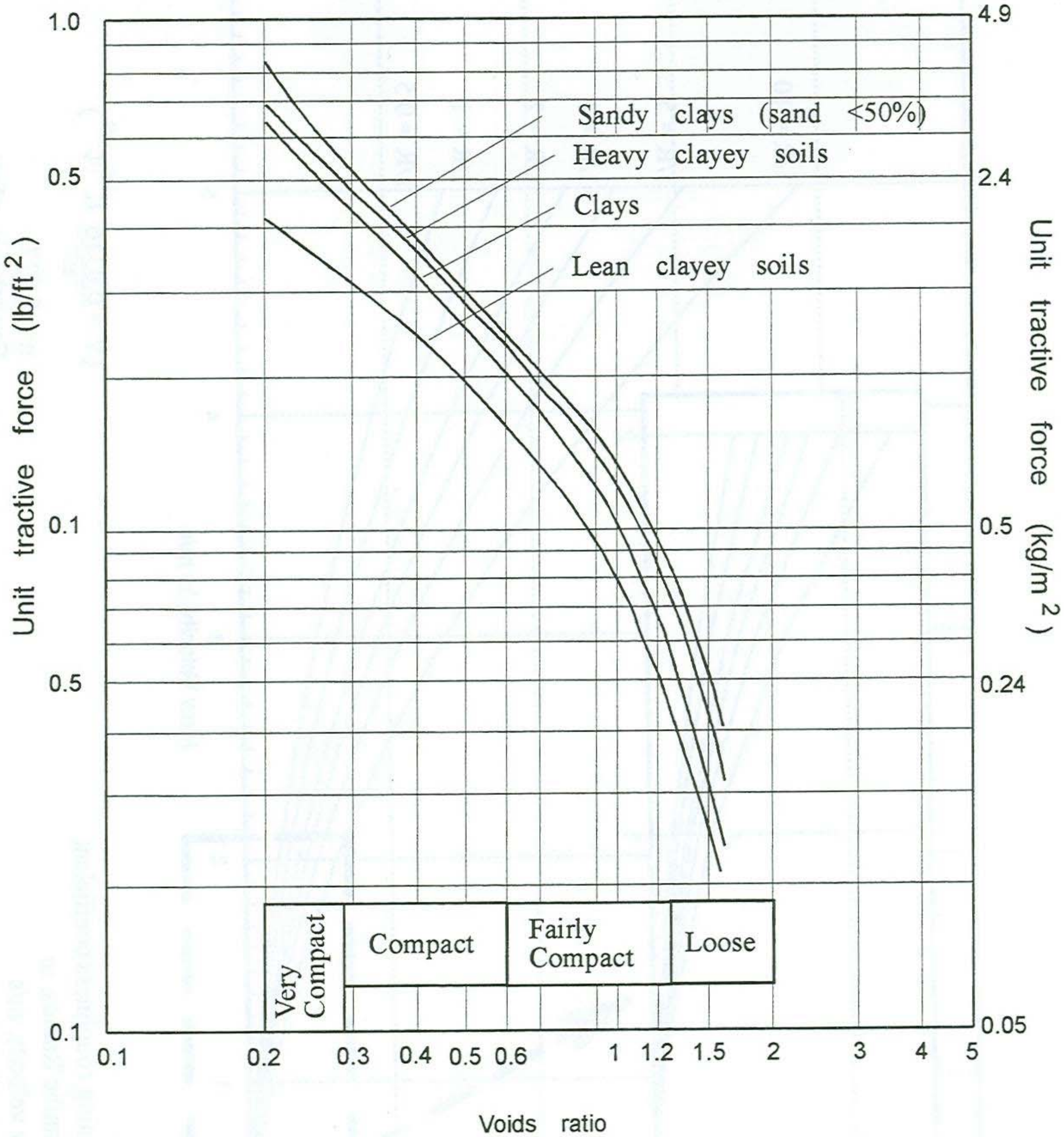


Source: F.C. Sobey (1939)

Design Chart 2.24: Tractive Force - Velocity Relationships



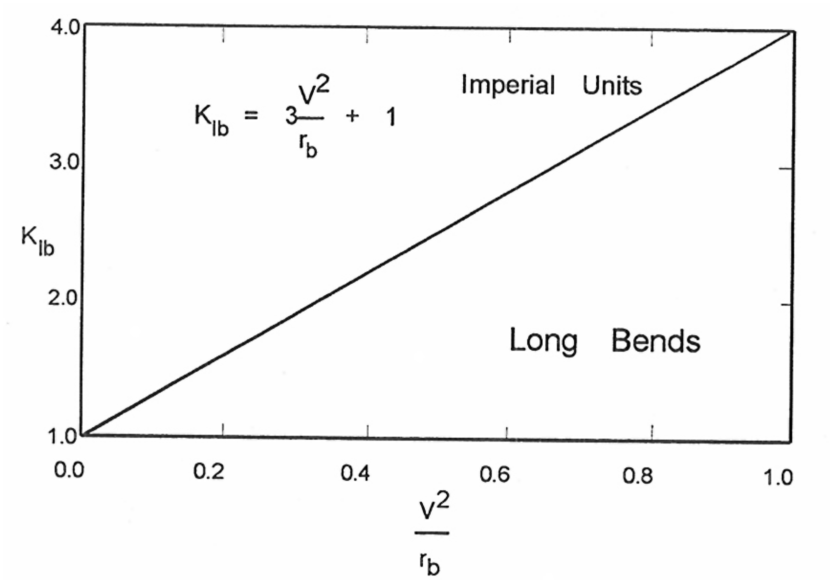
Design Chart 2.25: Permissible Unit Tractive Force



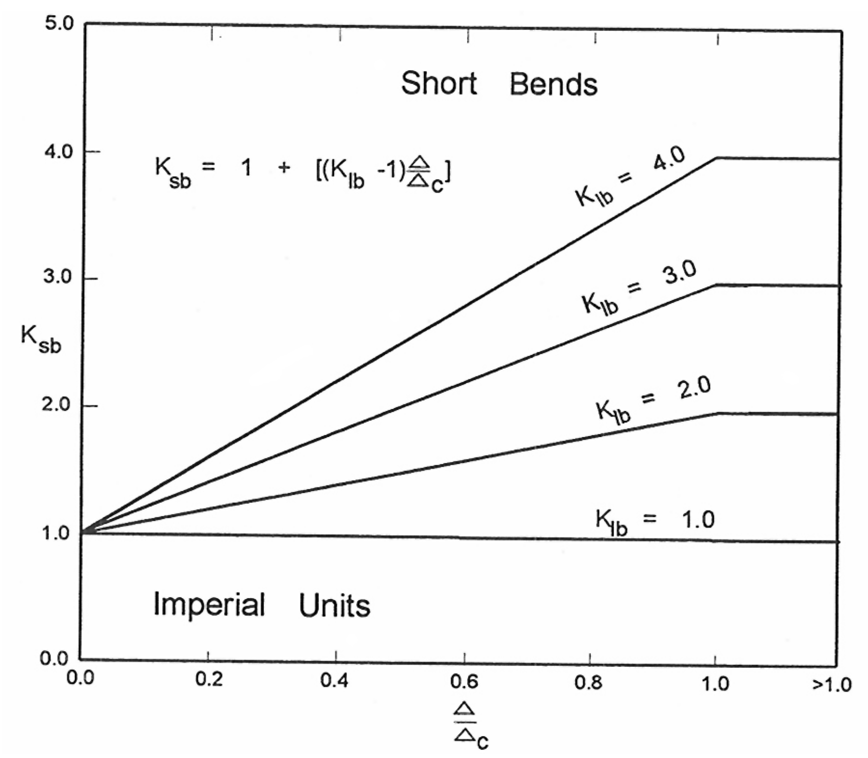
(b) Permissible Unit Tractive Force

Source: After Tentative Design Procedures for Riprap Lined Channels, 1970, A.G. Anderson, A.S. Paintal and J.T. Davenport, National Cooperative Highway Research Program Report 108.

Design Chart 2.26: Ratio of Shear in Long Bends to Straight Reach

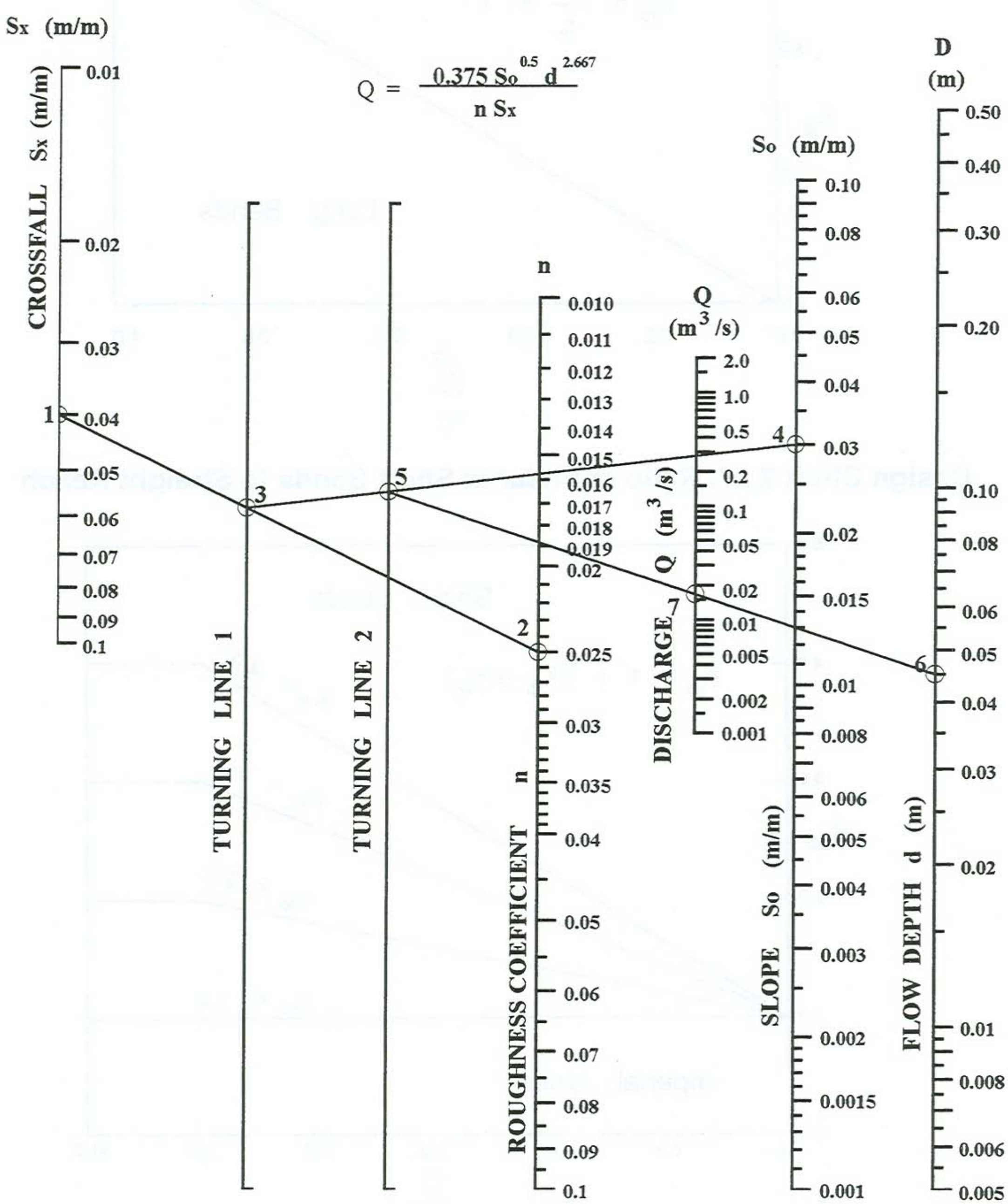


Design Chart 2.27: Ratio of Shear in Short Bends to Straight Reach

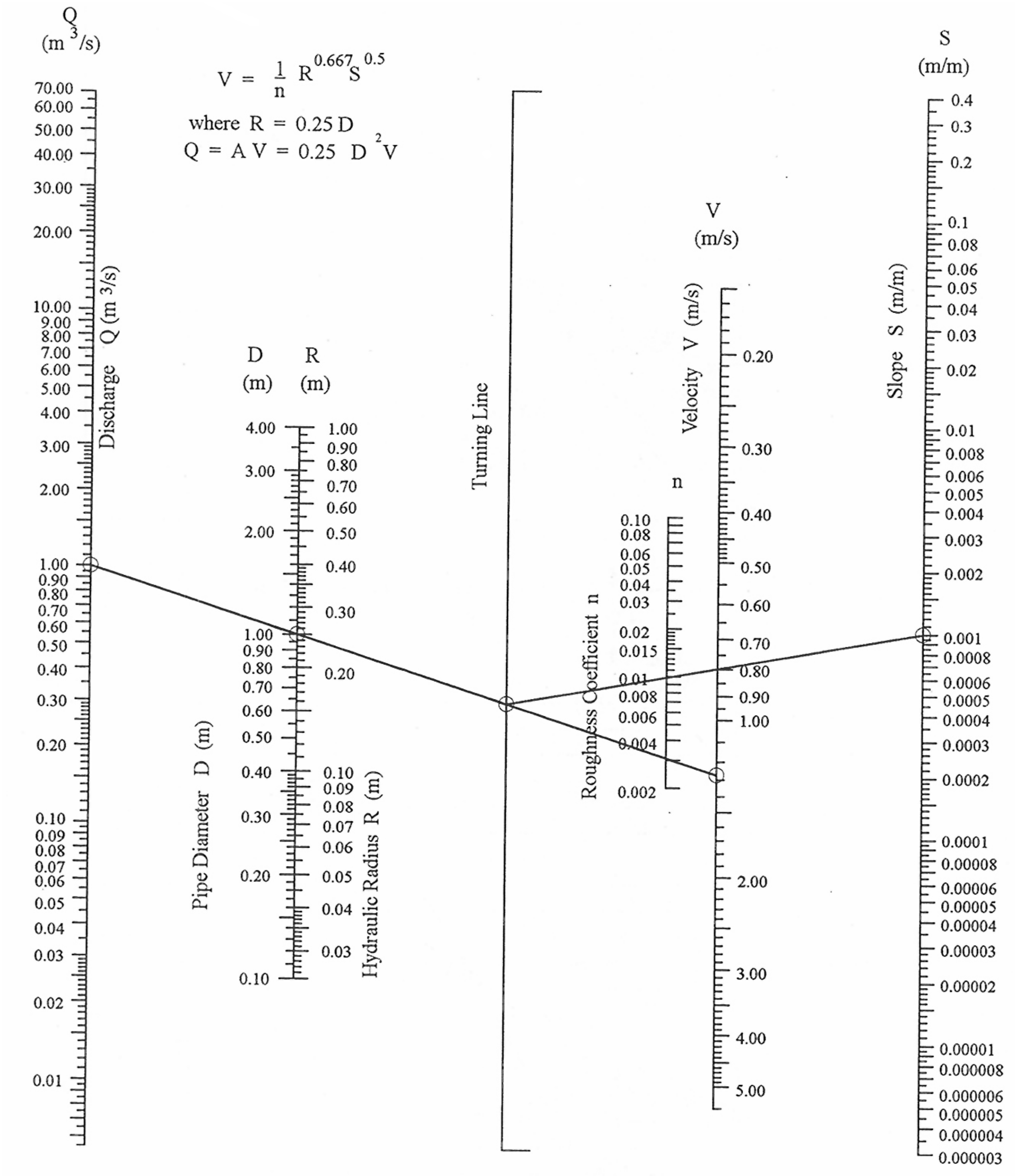


Source: After Tentative Design Procedures for Riprap Lined Channels, 1970, A.G. Anderson, A.S. Paintal and J.T. Davenport, National Cooperative Highway Research Program Report 108.

Design Chart 2.28: Nomograph: Triangular Channels

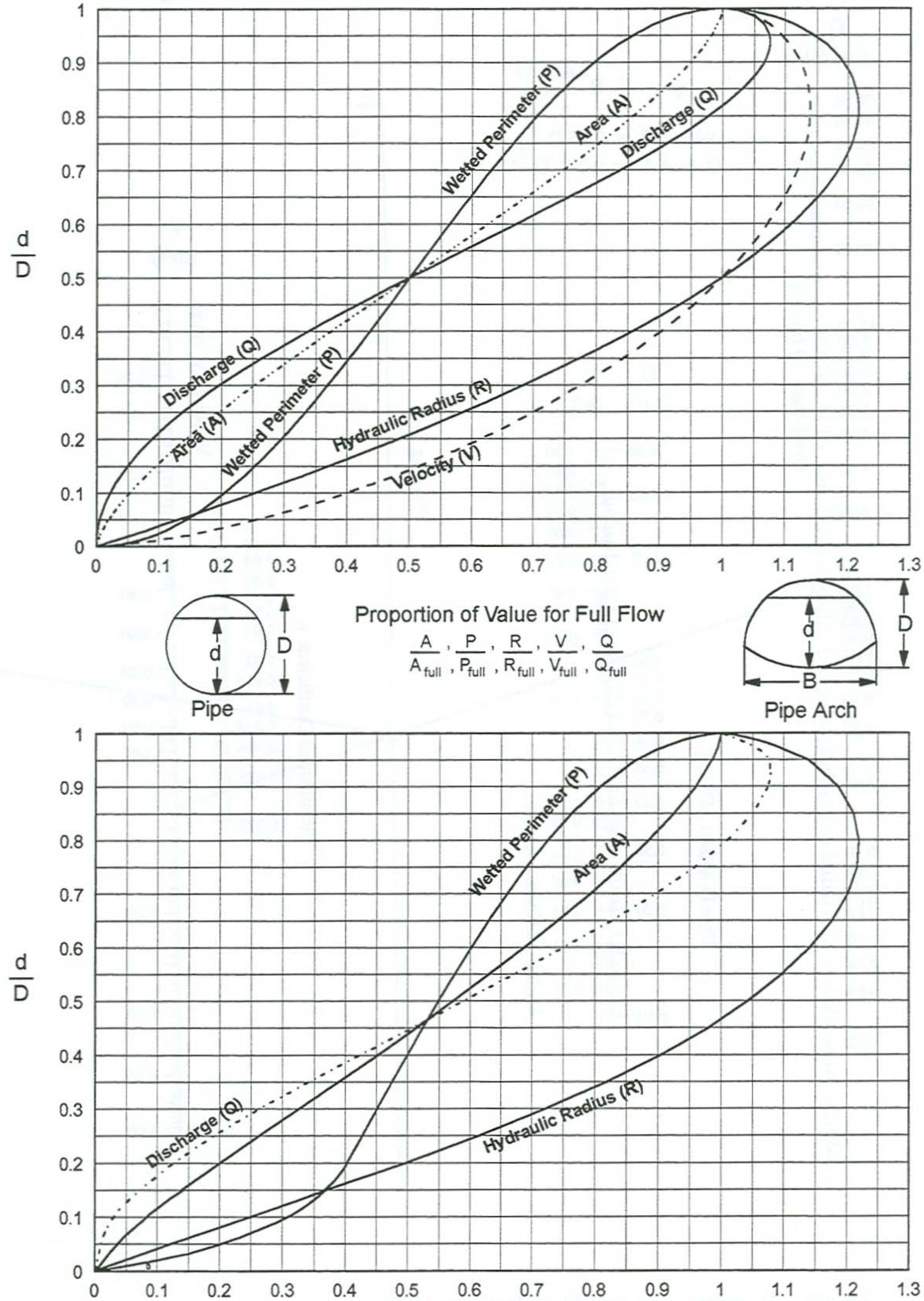


Design Chart 2.29: Nomograph: Circular Pipes - Flowing Full



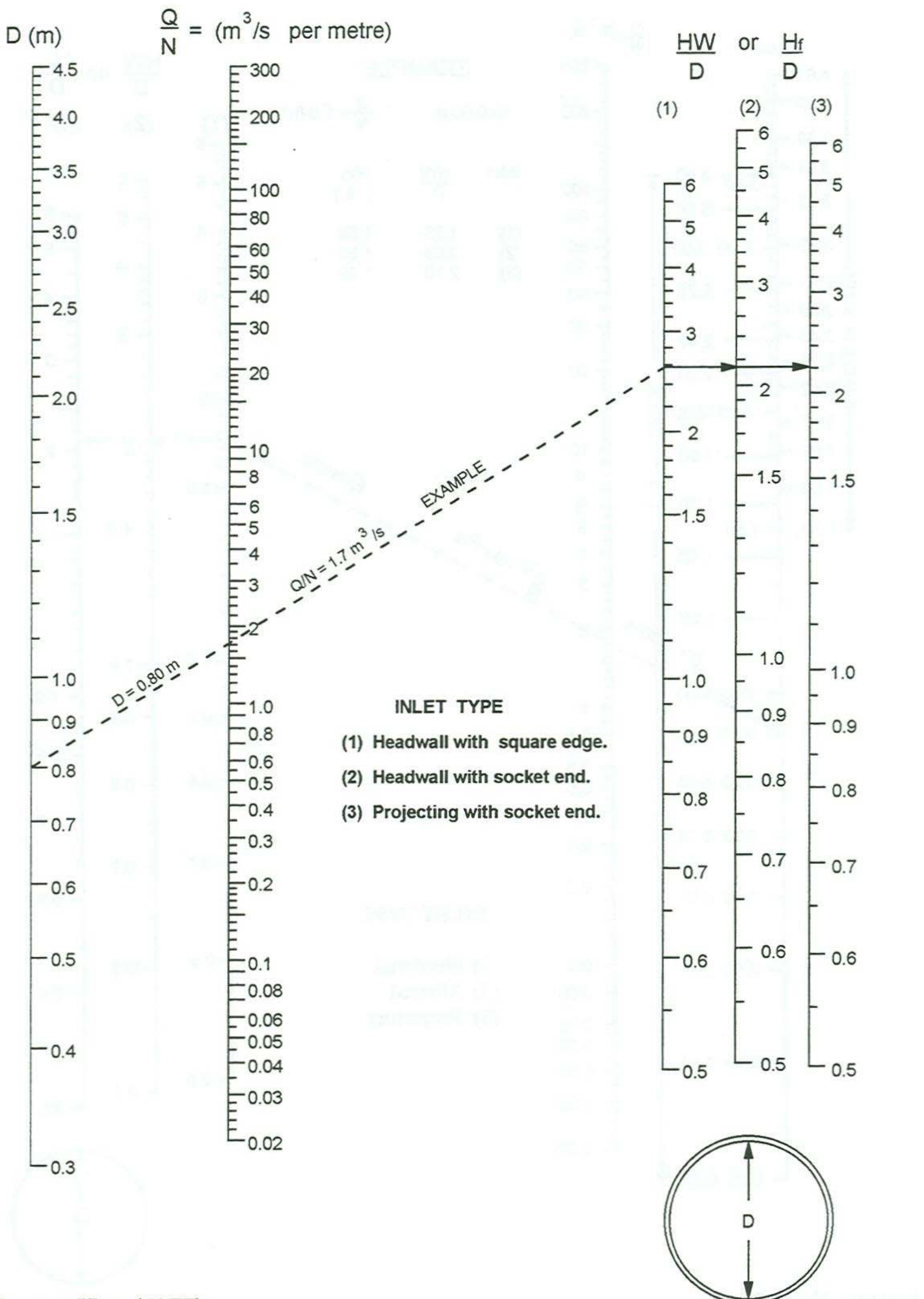
Source: American Iron and Steel Institute (1980)

Design Chart 2.30: Nomograph: Part-Full Flow for Pipes and Arches



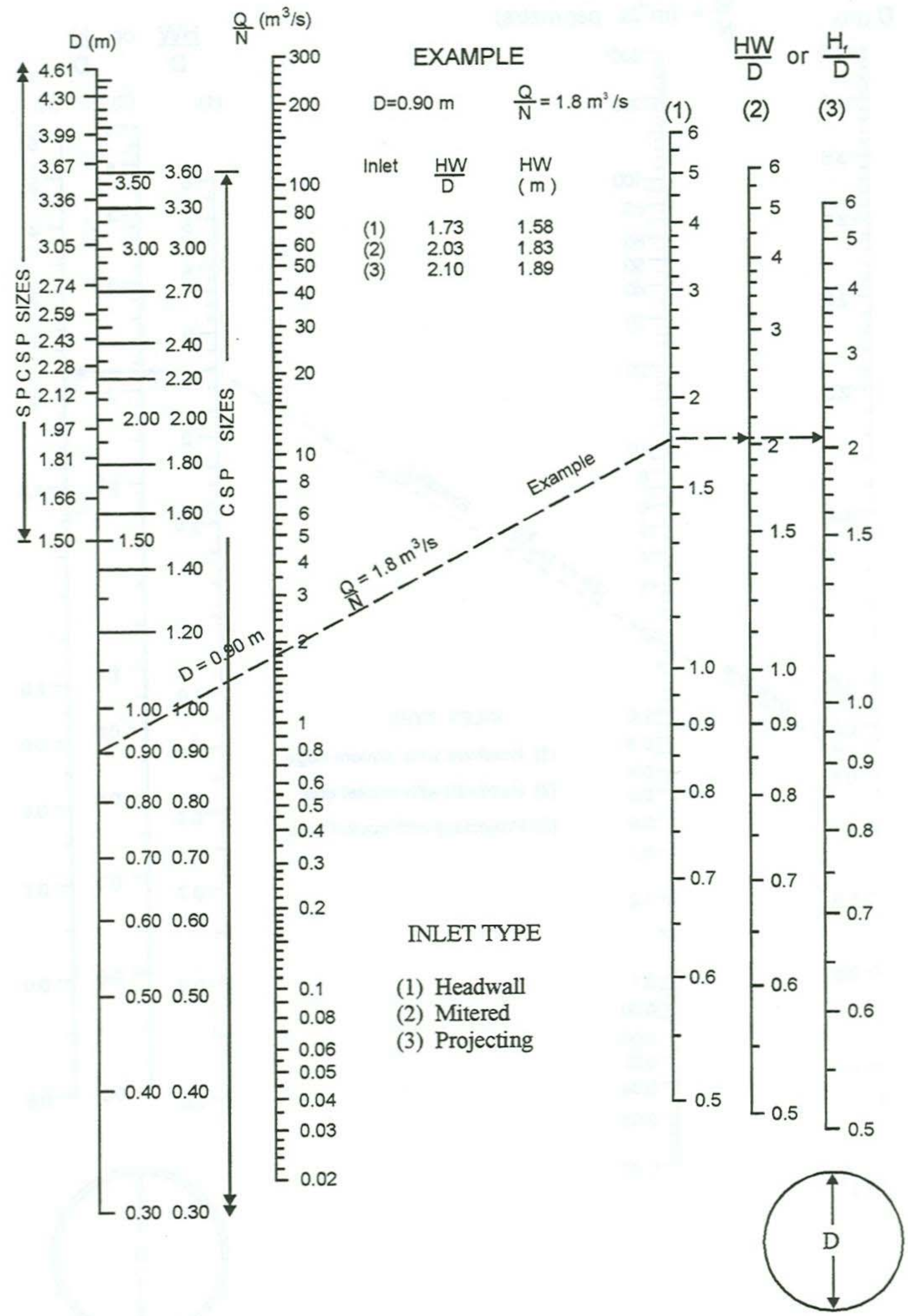
Source: American Iron and Steel Institute (1980)

Design Chart 2.31: Inlet Control: Circular Pipes



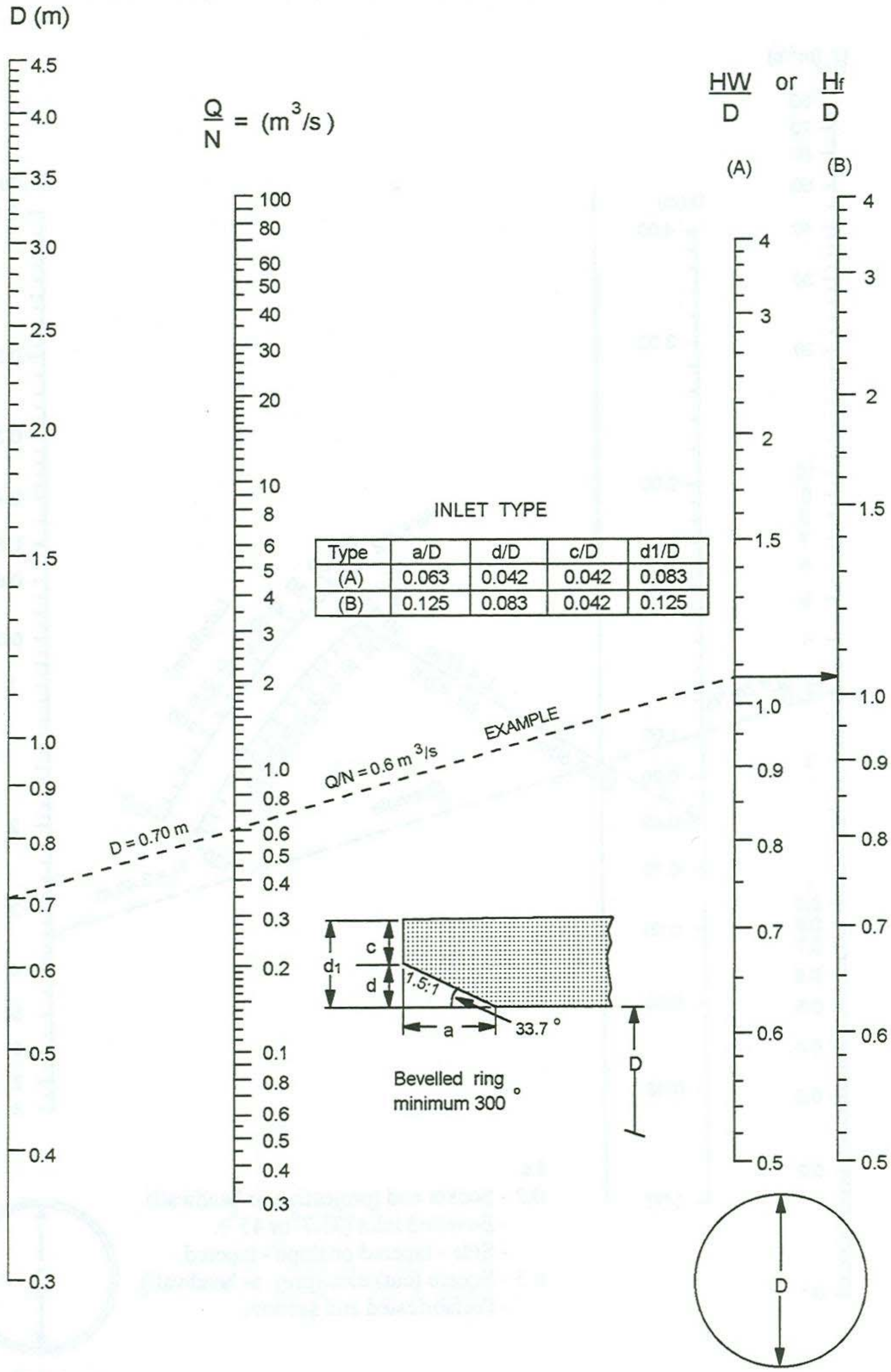
Source: Herr (1977)

Design Chart 2.32: Inlet Control: Circular CSP and SPCSP Culverts



Source: Herr (1977)

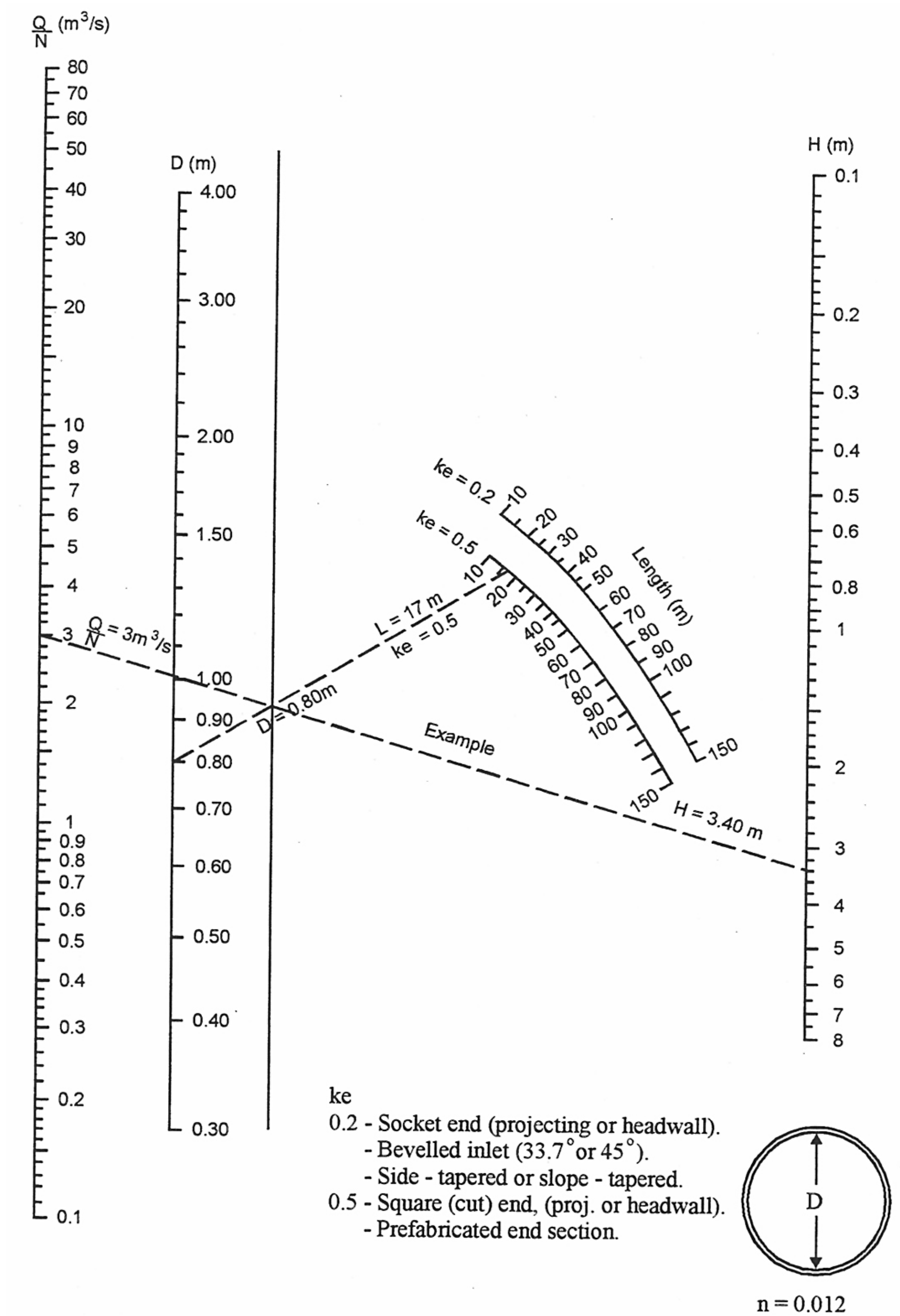
Design Chart 2.33: Inlet Control: Circular Culverts - Bevelled End



Type	a/D	d/D	c/D	d1/D
(A)	0.063	0.042	0.042	0.083
(B)	0.125	0.083	0.042	0.125

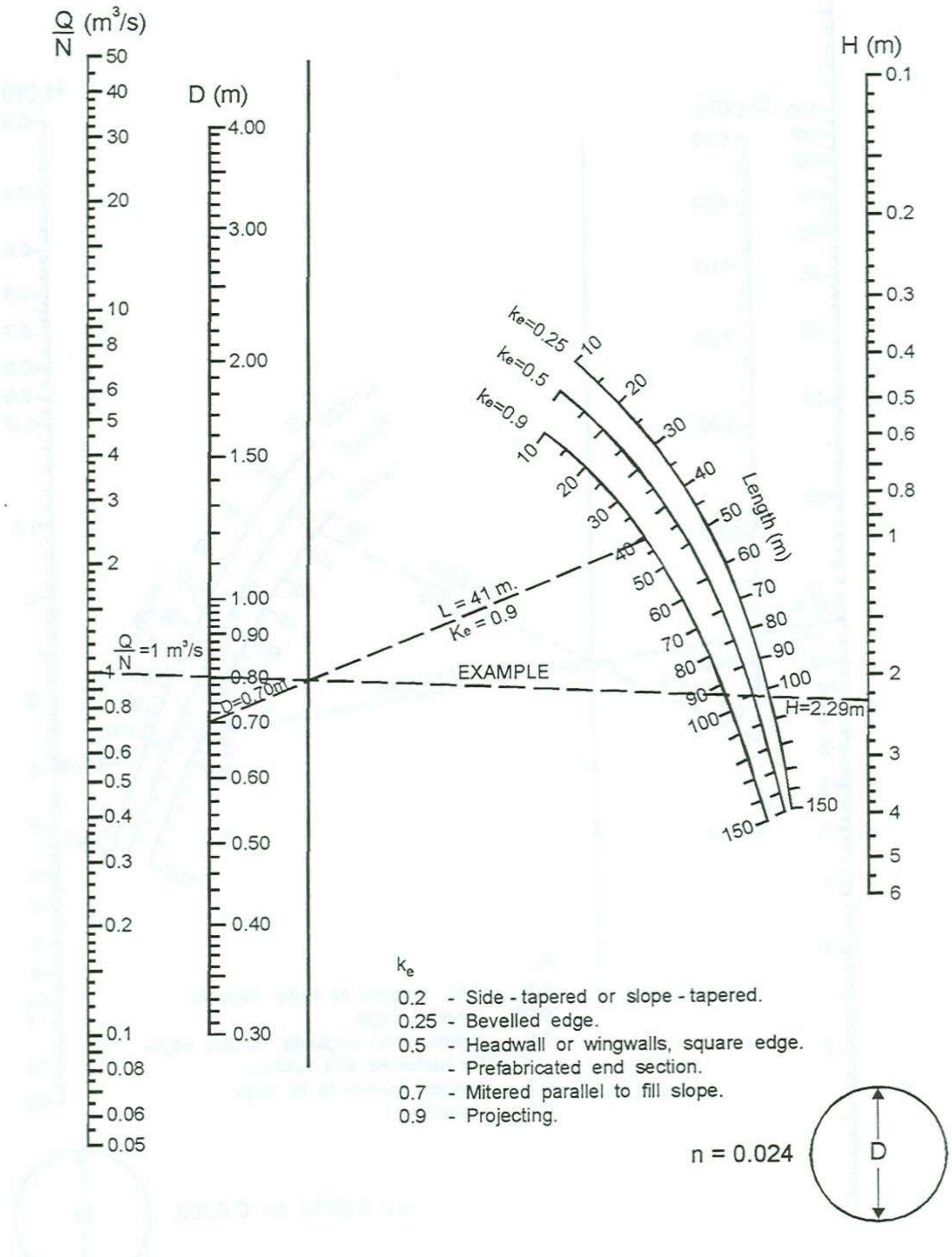
Source: Herr (1977)

Design Chart 2.34: Outlet Control: Concrete Circular Pipe/Culvert - Flowing Full



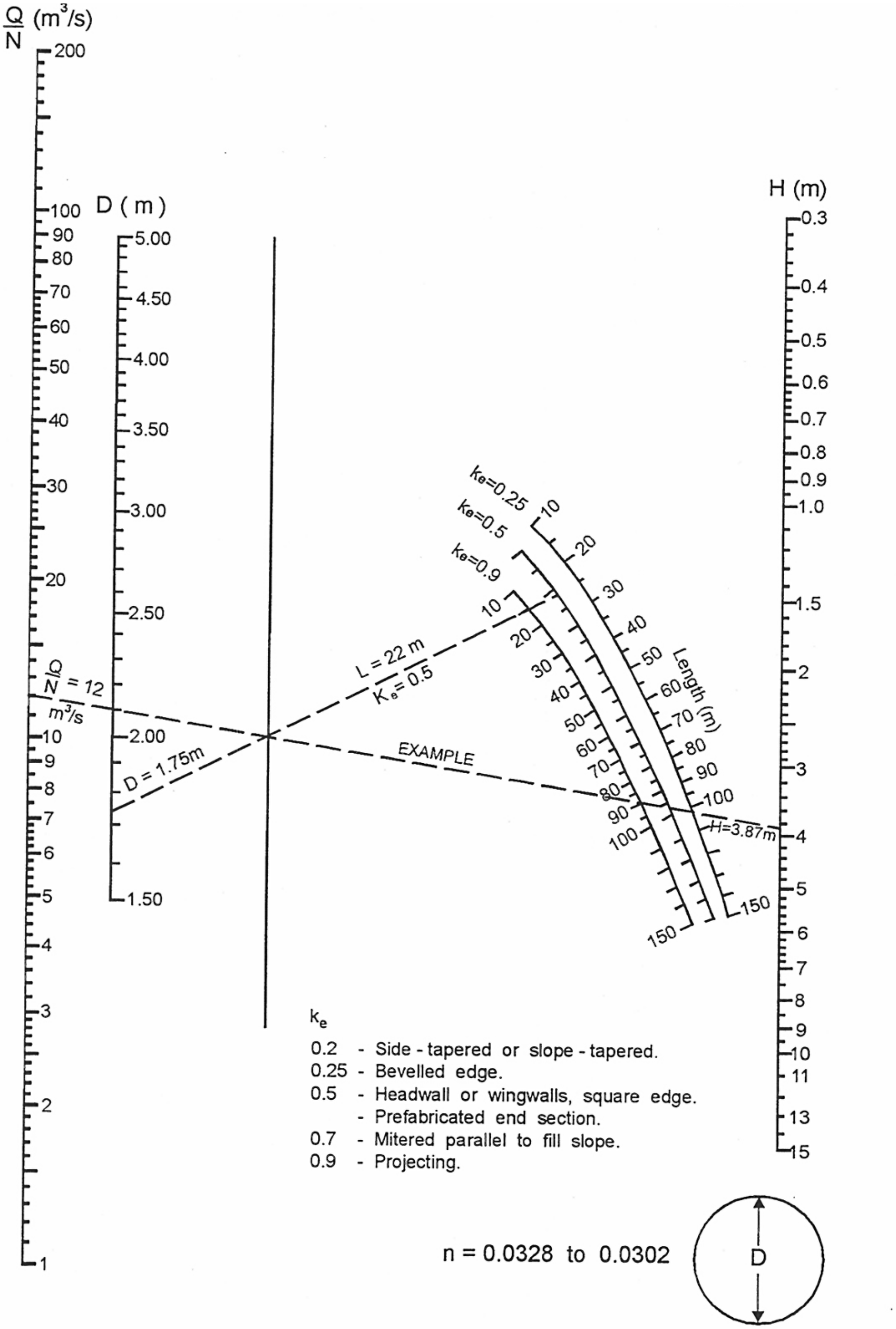
Source: Herr (1977)

Design Chart 2.35: Outlet Control: CSP Culvert - Flowing Full



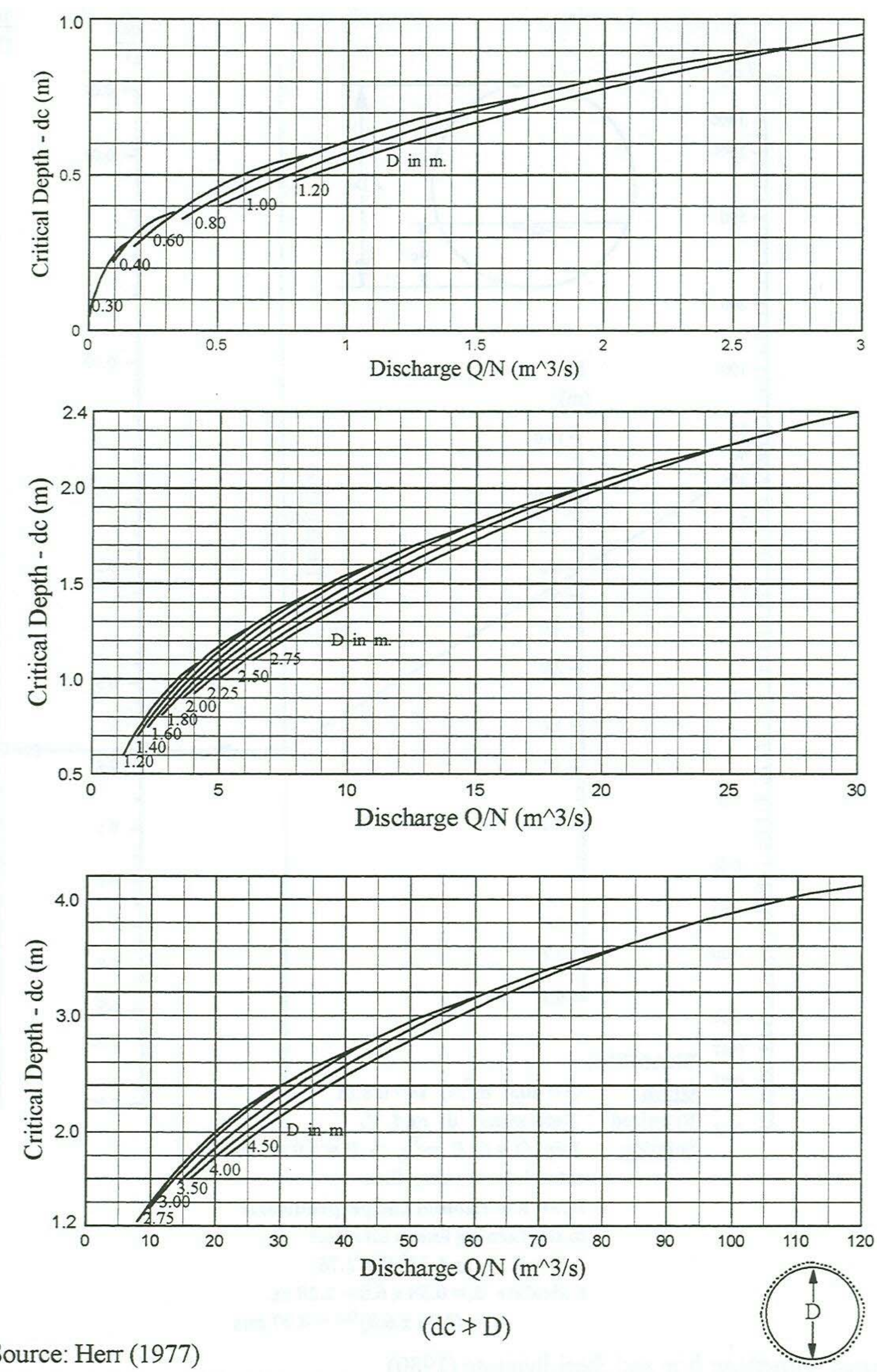
Source: Herr (1977)

Design Chart 2.36: Outlet Control: SPCSP Culvert - Flowing Full



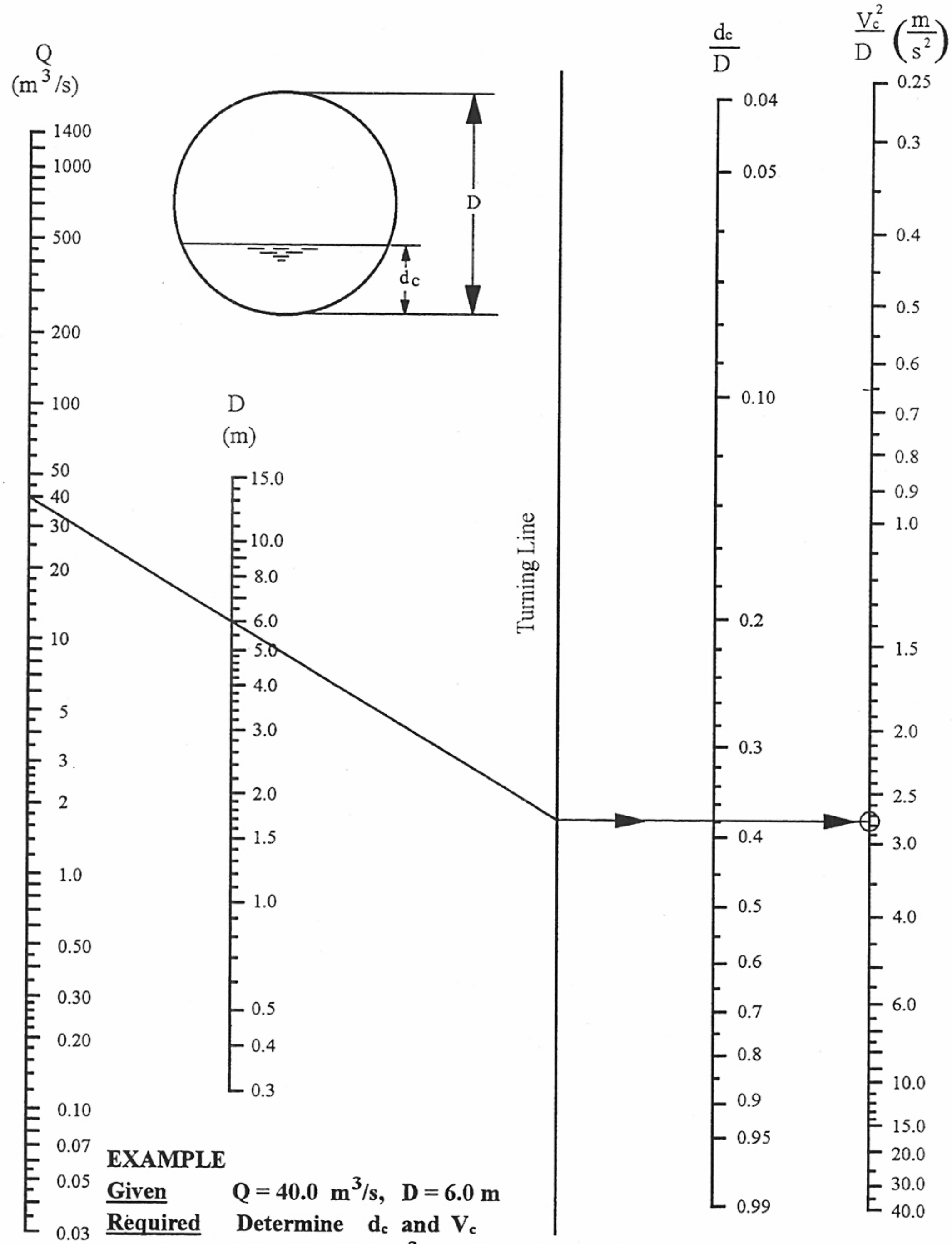
Source: Herr (1979)

Design Chart 2.37: Critical Depth Chart for Circular Pipes



Source: Herr (1977)

Design Chart 2.38: Critical Depth - Velocity relationships: Circular Pipes



EXAMPLE

Given Q = 40.0 m^3/s, D = 6.0 m

Required Determine d_c and V_c

Solution Join Q = 40.0 m^3/s to D = 6.0 m and extend to turning line.
Draw a horizontal line perpendicular to the turning line to intersect $d_c/D = 0.38$ and $V_c^2/D = 2.76$
Calculate $d_c = 0.38 \times 6.0 = 2.28$ m.
$V_c = (2.76 \times 6.0)^{0.5} = 4.07$ m/s

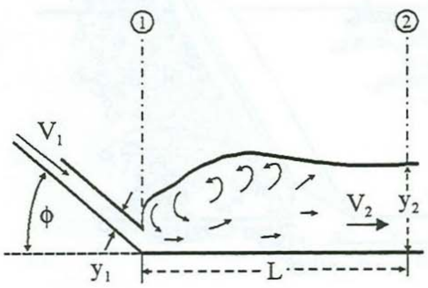
Source: American Iron and Steel Institute

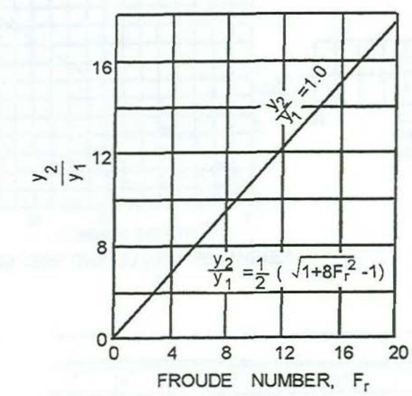
Design Chart 2.39: USBR Energy Dissipator Type I/Vertical Drop

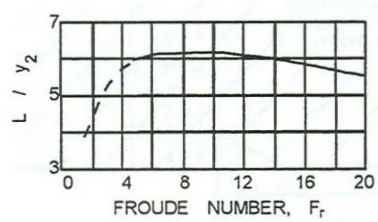
Jump occurs on flat floor with no chute blocks, baffle piers or end sill in basin. Usually not a practical basin because of excessive length.

Elements and characteristics of jumps for complete range of Froude Numbers.

For use in stilling basins with Froude Numbers less than 2.5.



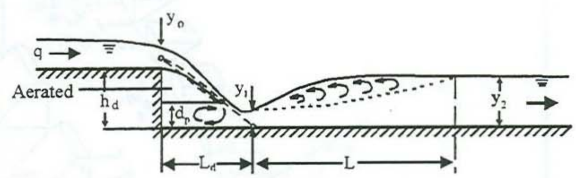




VERTICAL DROP

For small drops of up to 1.0 m.

Jump starts at a distance of L_d, which varies with vertical drop distance, h_d, and is influenced by nappe pool depth, d_p.



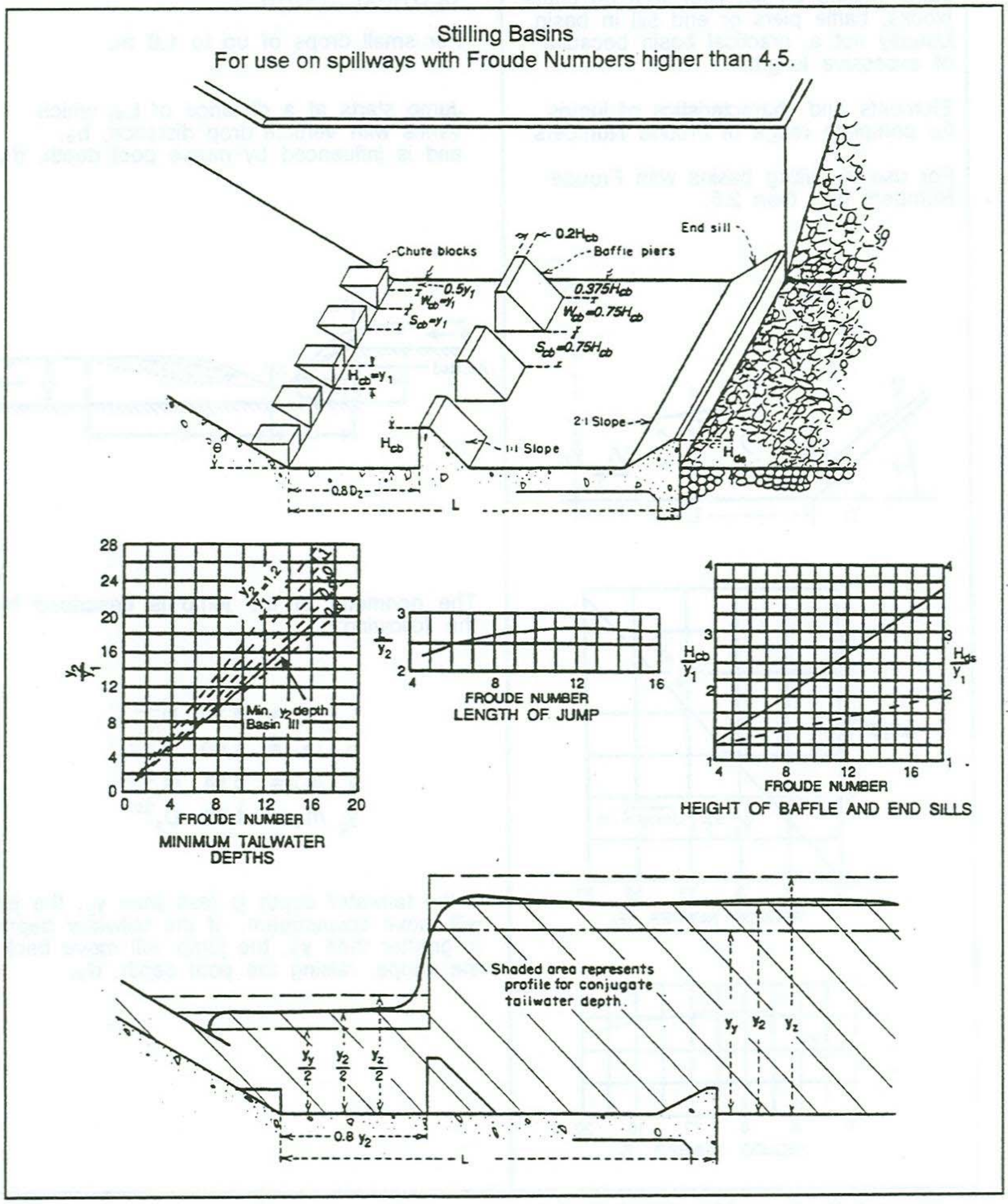
The geometry of the jump is described by the following:

$$L_d / h_d = 4.30 \ D_d^{0.27}$$
$$d_p / h_d = 1.00 \ D_d^{0.22}$$
$$y_1 / h_d = 0.54 \ D_d^{0.425}$$
$$y_2 / h_d = 1.66 \ D_d^{0.27}$$

If the tailwater depth is less than y_2, the jump will move downstream. If the tailwater depth is greater than y_2, the jump will move back on the nappe, raising the pool depth, d_p.

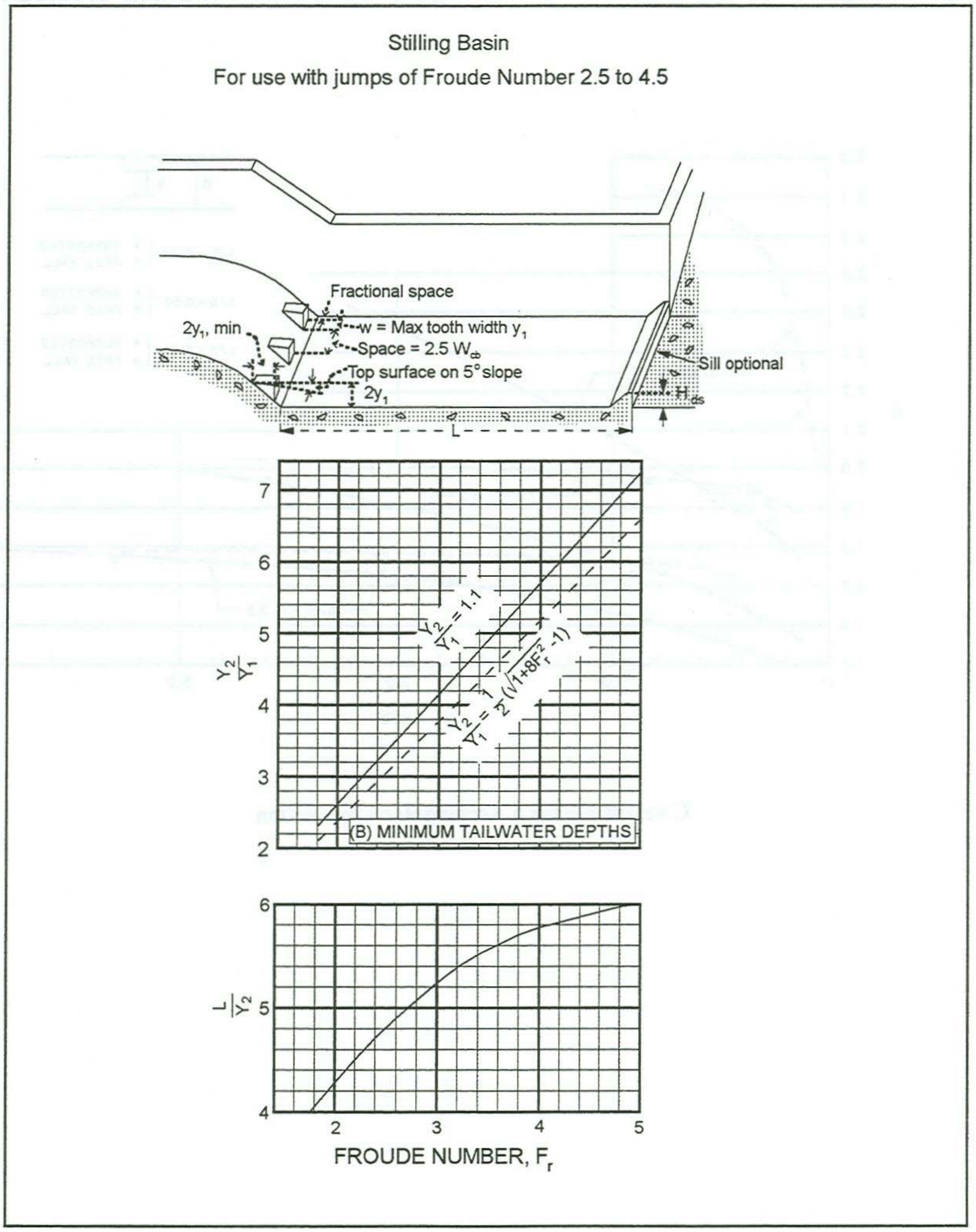
Source: A.J. Peterka (1974), RTAC Drainage Manual Volume 1, 1962.

Design Chart 2.40: USBR Energy Dissipator, Type III



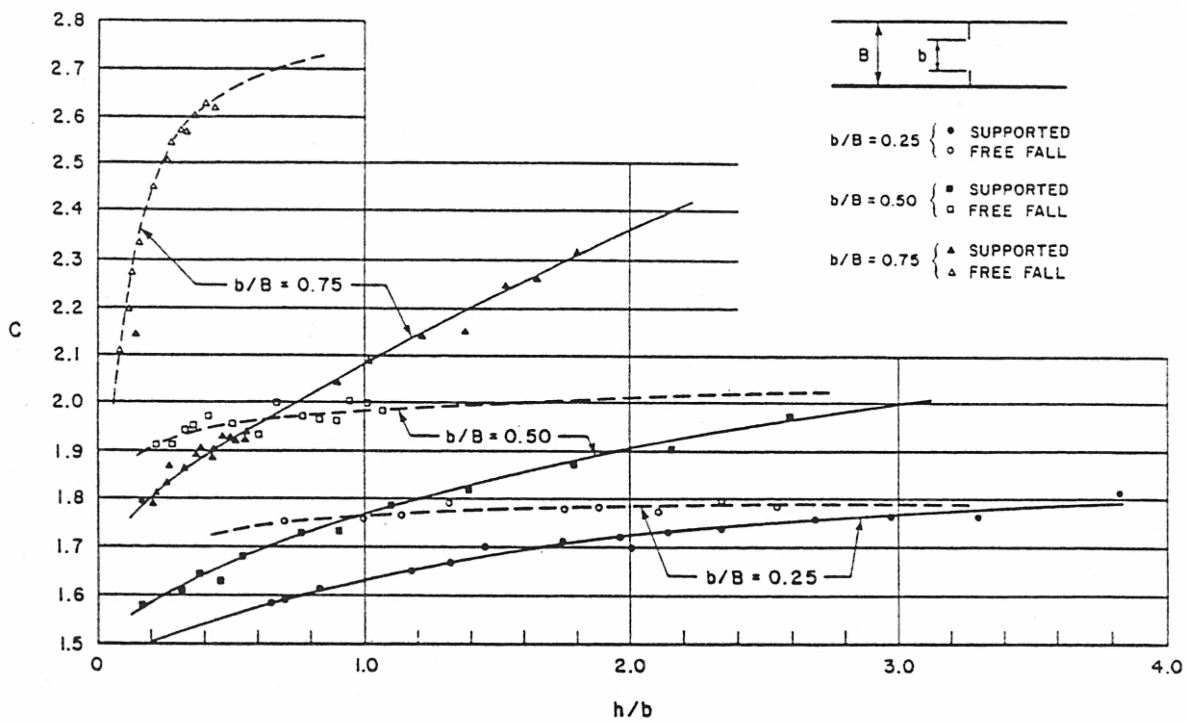
Source: A.J. Peterka (1974)

Design Chart 2.41: USBR Energy Dissipator, Type IV



Source: A.J. Peterka (1974)

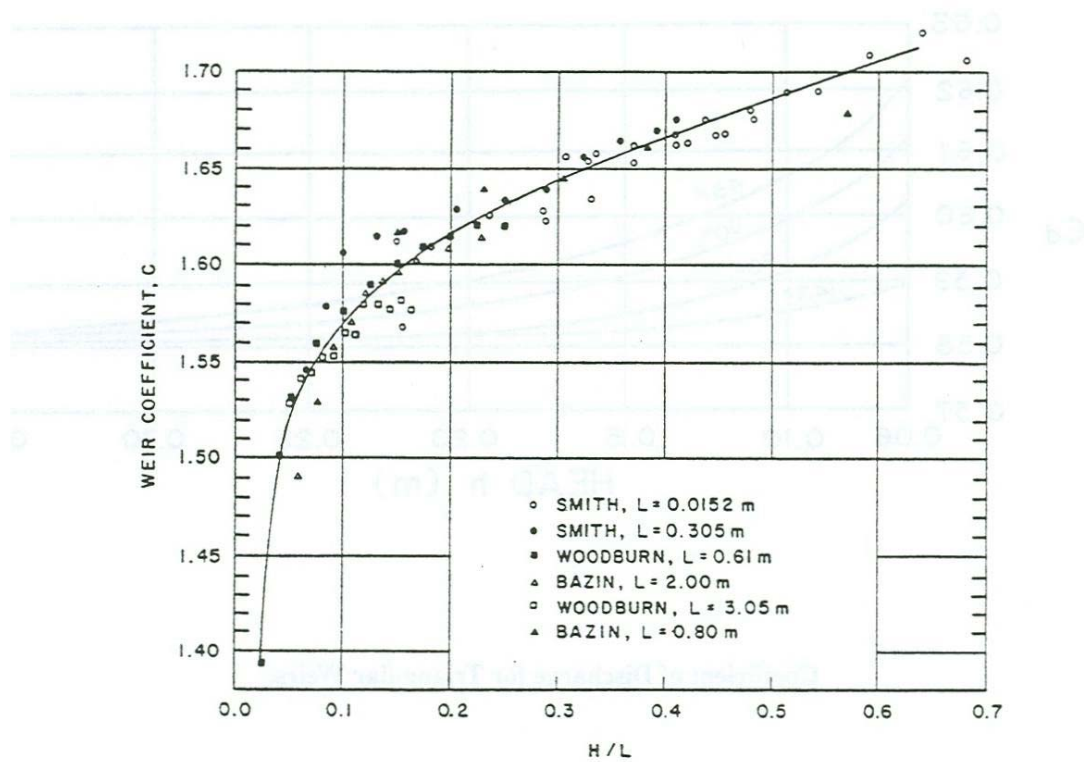
Design Chart 2.42: C vs h/b for Rectangular Contraction of Sharp Crested Weirs



C versus h/b for a Rectangular Contraction

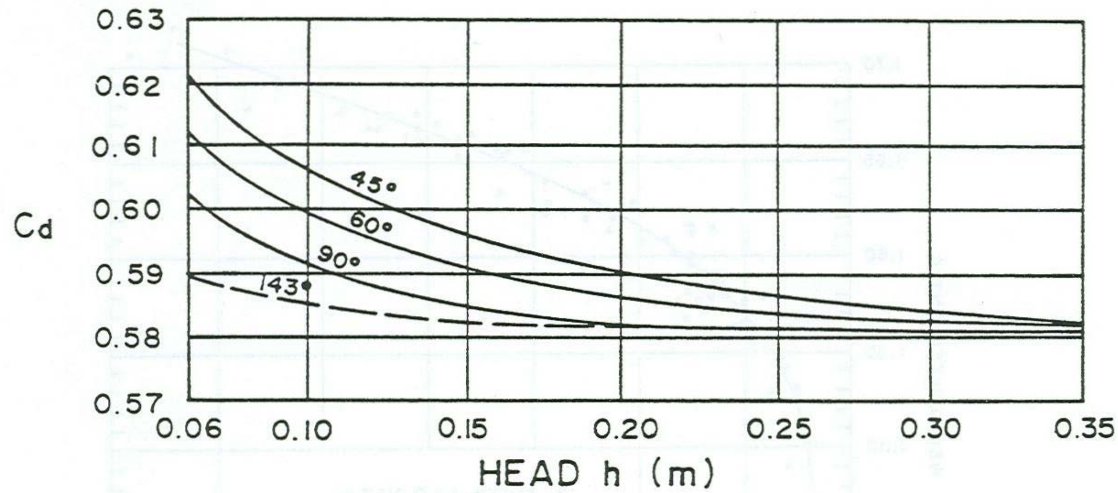
Source: C.D. Smith (1985)

Design Chart 2.43: Coefficient of Discharge for Rectangular Broad Crested Weir



Source: C.D. Smith (1985)

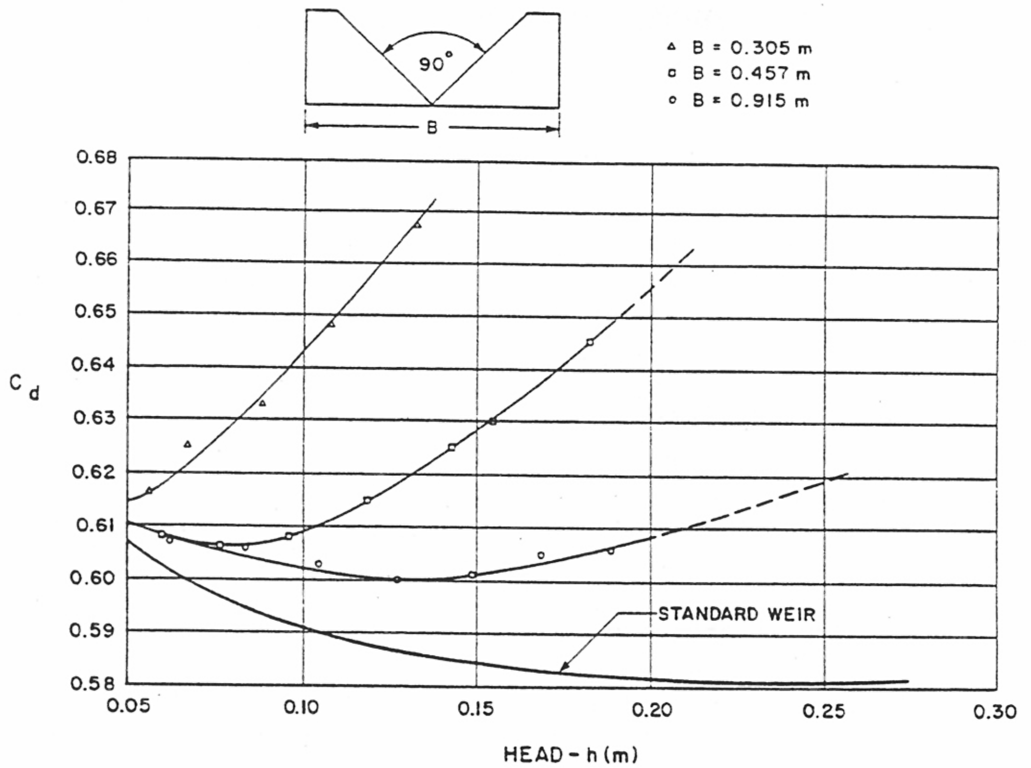
Design Chart 2.44: Coefficient of Discharge for Triangular Sharp Crested Weir



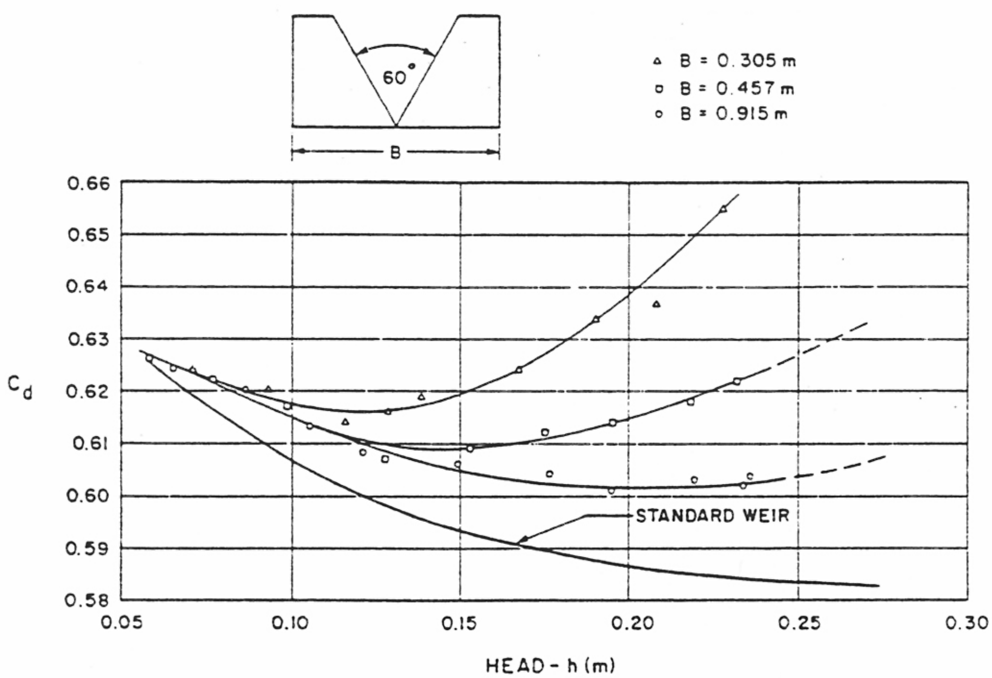
Coefficient of Discharge for Triangular Weirs.

Source: C.D. Smith (1985)

Design Chart 2.45: Coefficient of Discharge for 90° & 60° V-notch Contraction



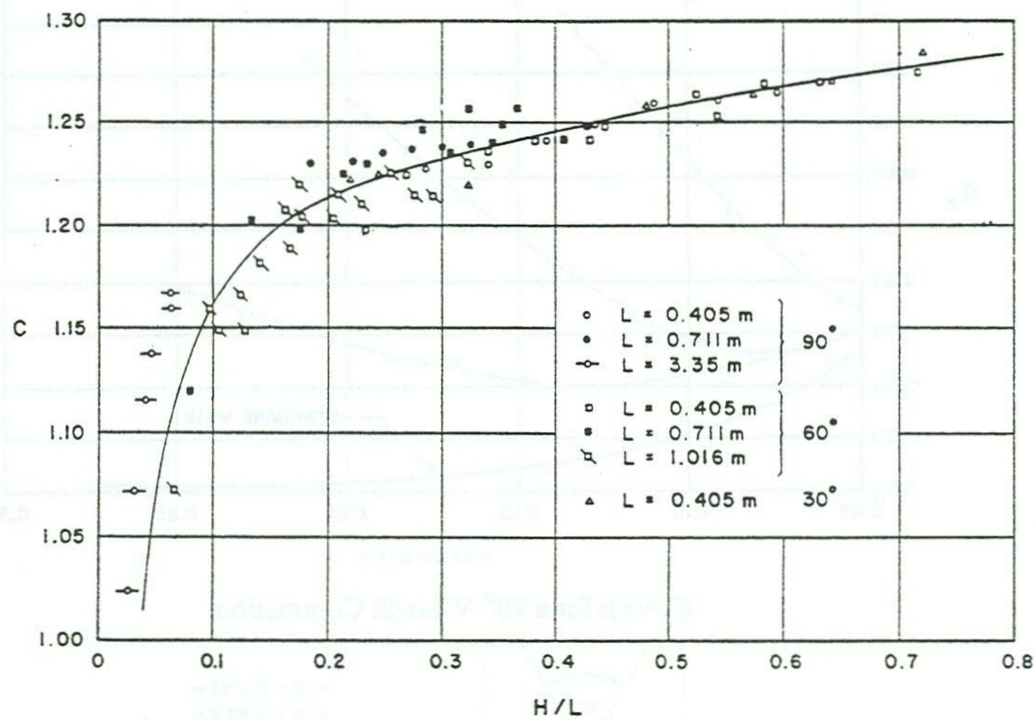
C_d vs h for a 90° V-notch Contraction



C_d vs h for a 60° V-notch Contraction

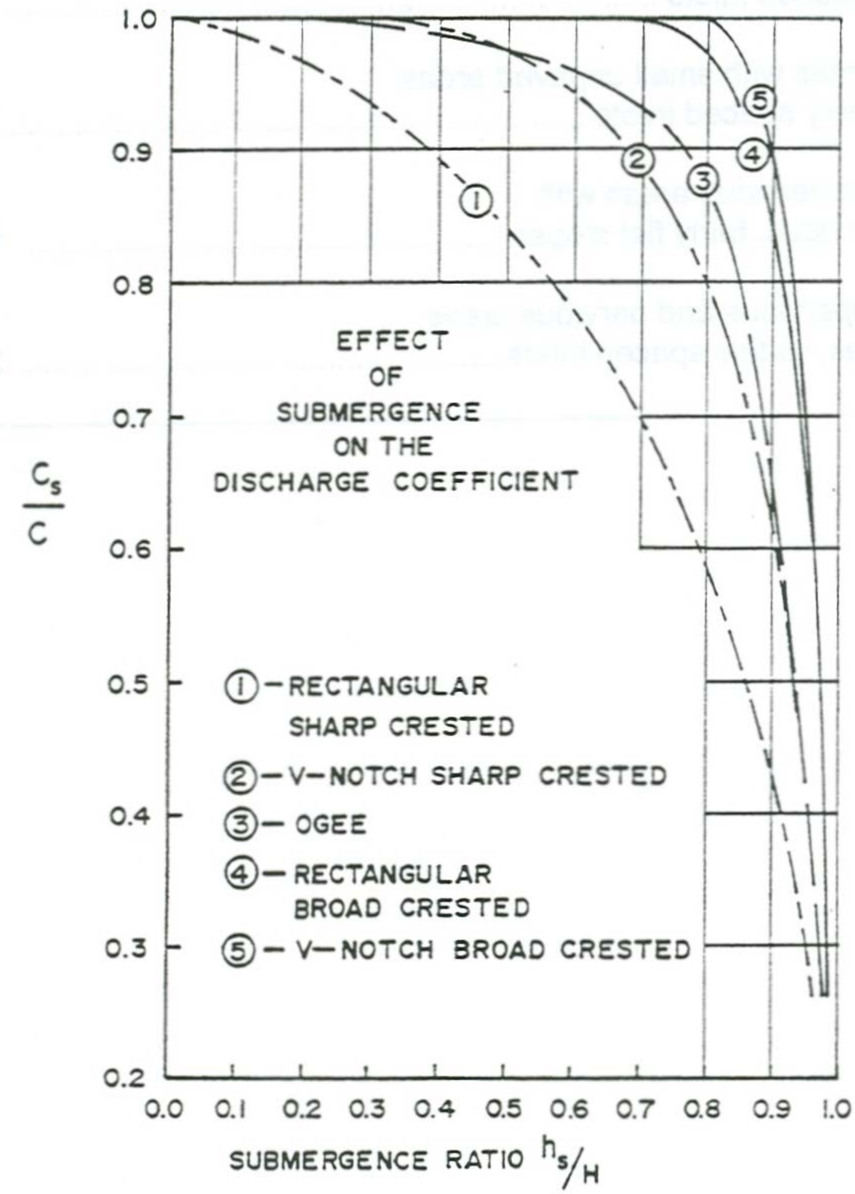
Source: C.D. Smith (1985)

Design Chart 2.46: Coefficient of Discharge for Triangular Broad Crested Weir



Source: C.D. Smith (1985)

Design Chart 2.47: Effect of Submergence on Weir Coefficient

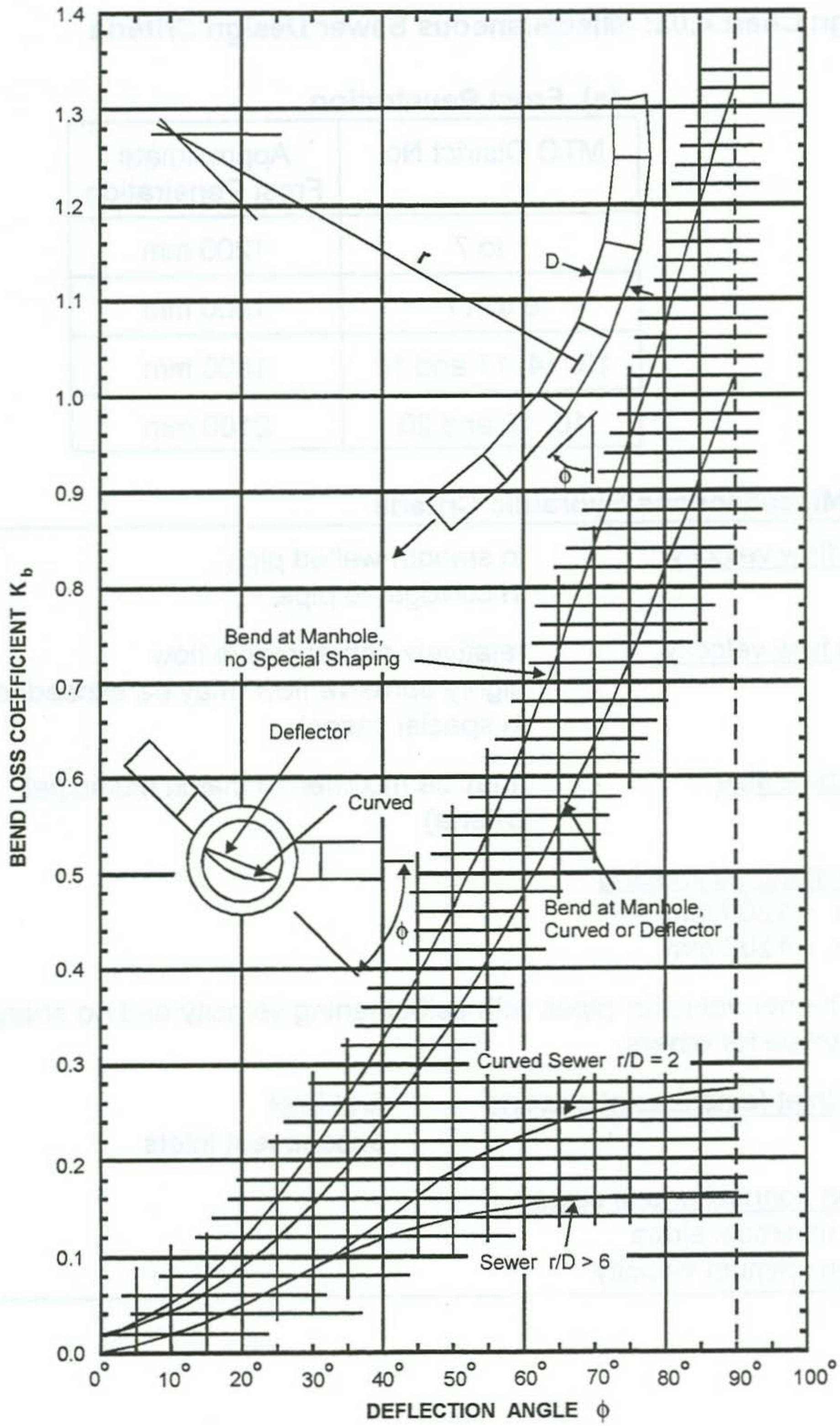


Source: C.D. Smith (1985)

Design Chart 4.01: Sewer Inlet Times

Paved areas draining directly to closely spaced inlets	5 to 10 min.
Paved areas with small unpaved areas, more widely spaced inlets	10 min.
Largely impervious areas with some pervious, fairly flat slopes	10 to 15 min.
Mixed impervious and pervious areas, flat grades, widely spaced inlets	20 to 30 min.

Design Chart 4.02: Sewer Bend Loss Coefficients



Source: American Iron and Steel Institution (1980)

Design Chart 4.03: Miscellaneous Sewer Design Criteria

(a) Frost Penetration

MTO District No.	Approximate Frost Penetration
1 to 7	1200 mm
8 to 11	1500 mm
13, 14, 17 and 18	1800 mm
16, 19 and 20	2100 mm

(b) Miscellaneous Hydraulic Criteria

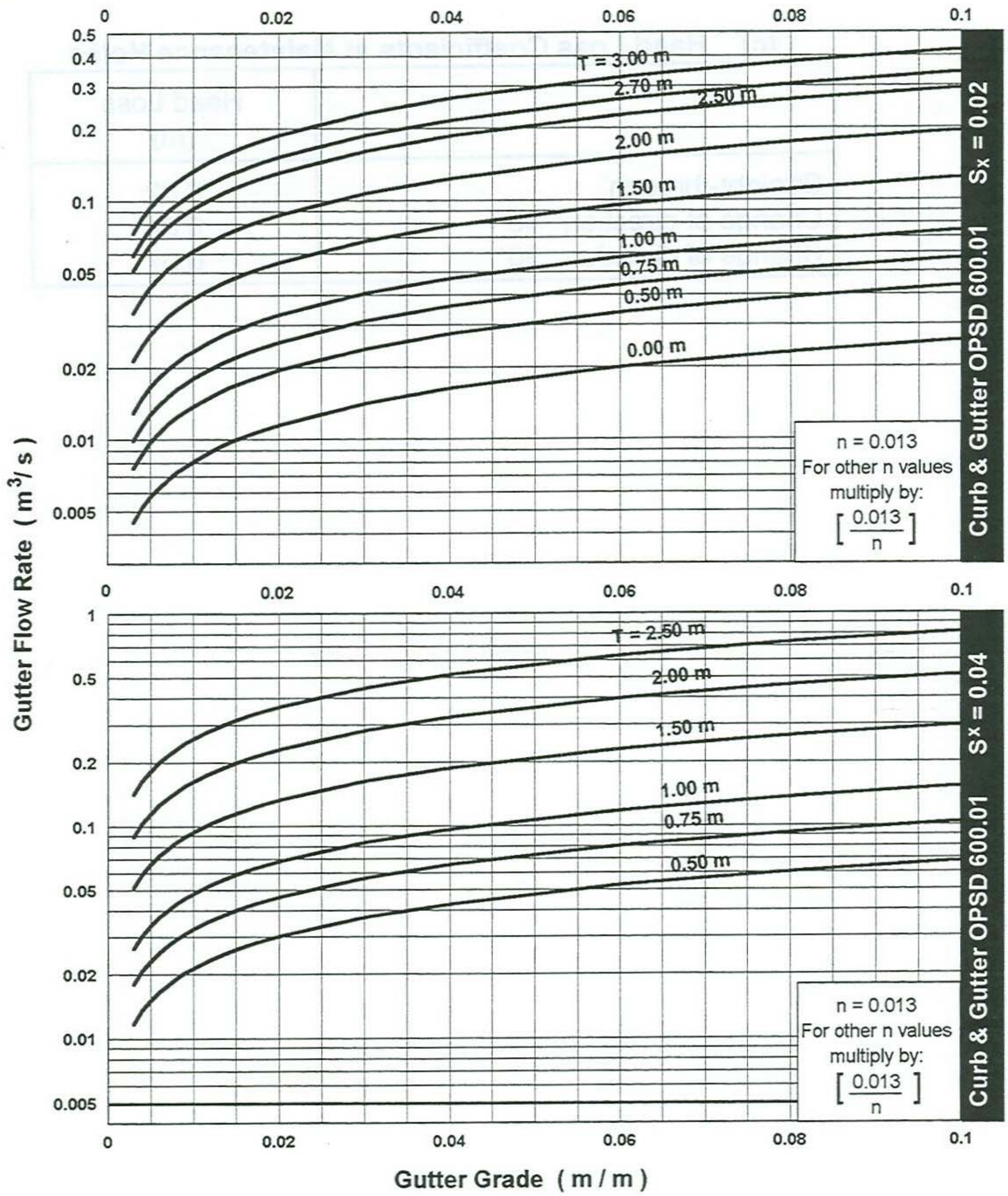
Minimum flow velocity	-	in smooth-walled pipe	0.75 m/s
	-	in corrugated pipe	0.9 m/s
Maximum flow velocity	-	relatively non-abrasive flow	10.0 m/s
	-	highly abrasive flow (may be exceeded in special cases)	5.0 m/s
Minimum pipe size	-	(may be modified to match municipal	300 mm
criteria)			
Maximum manhole spacing			
Pipe diam. < 1200 mm			100-150 m
Pipe diam. ∃ 1200 mm			200-350 m*
* Use the higher value for pipes with self-cleaning velocity and no sharp bends, and the lower value for others.			
Maximum inlet (catchbasin) spacing	-	first inlet	150 m
	-	subsequent inlets	100-150 m
Catchbasin connections to sewers			
Desirable minimum slope			0.015 m/m
Desirable mimimum velocity			1.5 m/m

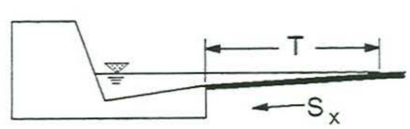
Design Chart 4.03: Miscellaneous Sewer Design Criteria (Continued)

(c) Head Loss Coefficients at Maintenance Holes

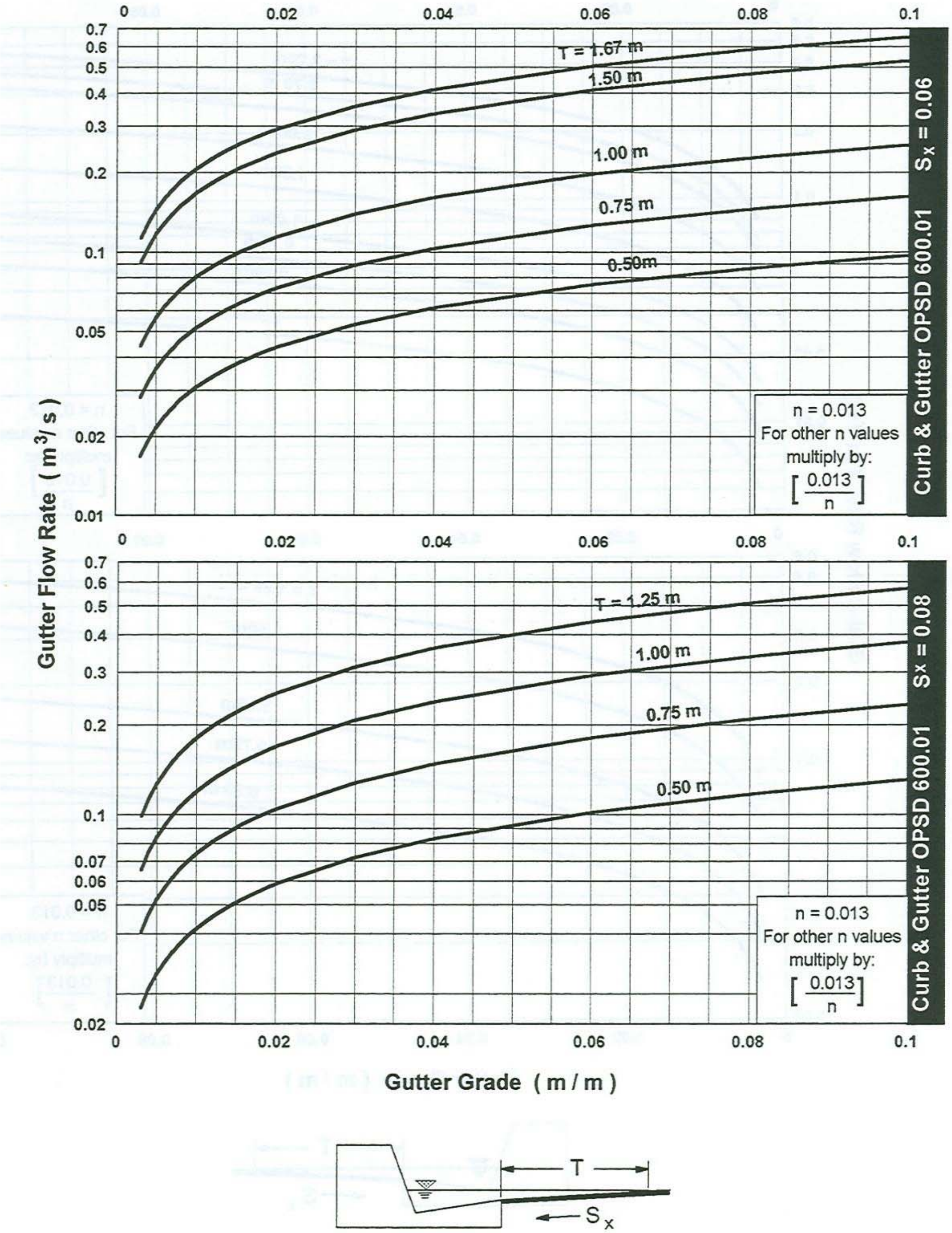
	Head Loss (m)
Straight-through	0.02
Change of direction 45E	0.04
Change of direction 90E	0.07

Design Chart 4.04: Gutter Flow Rate - Curb & Gutter OPSD 600.01

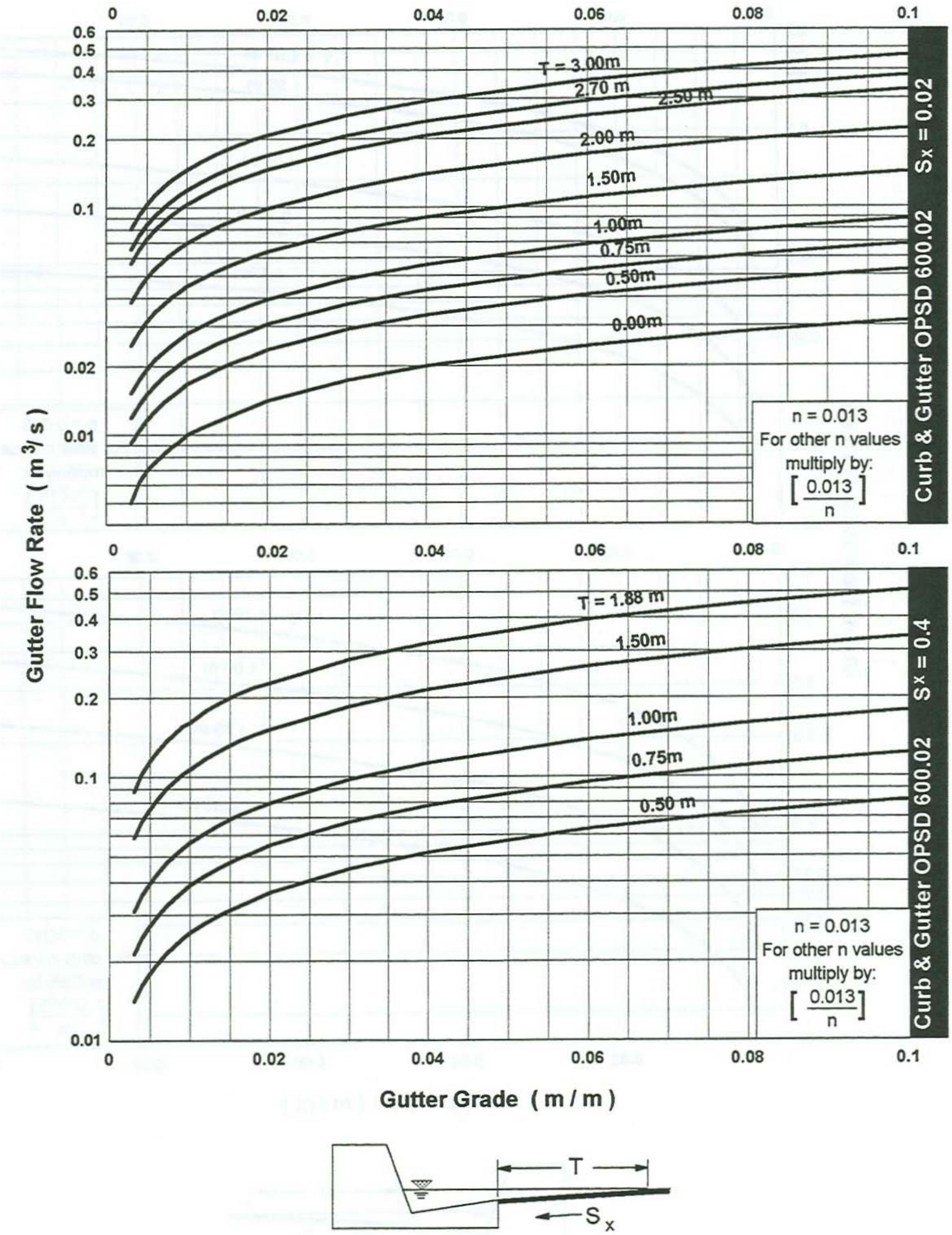




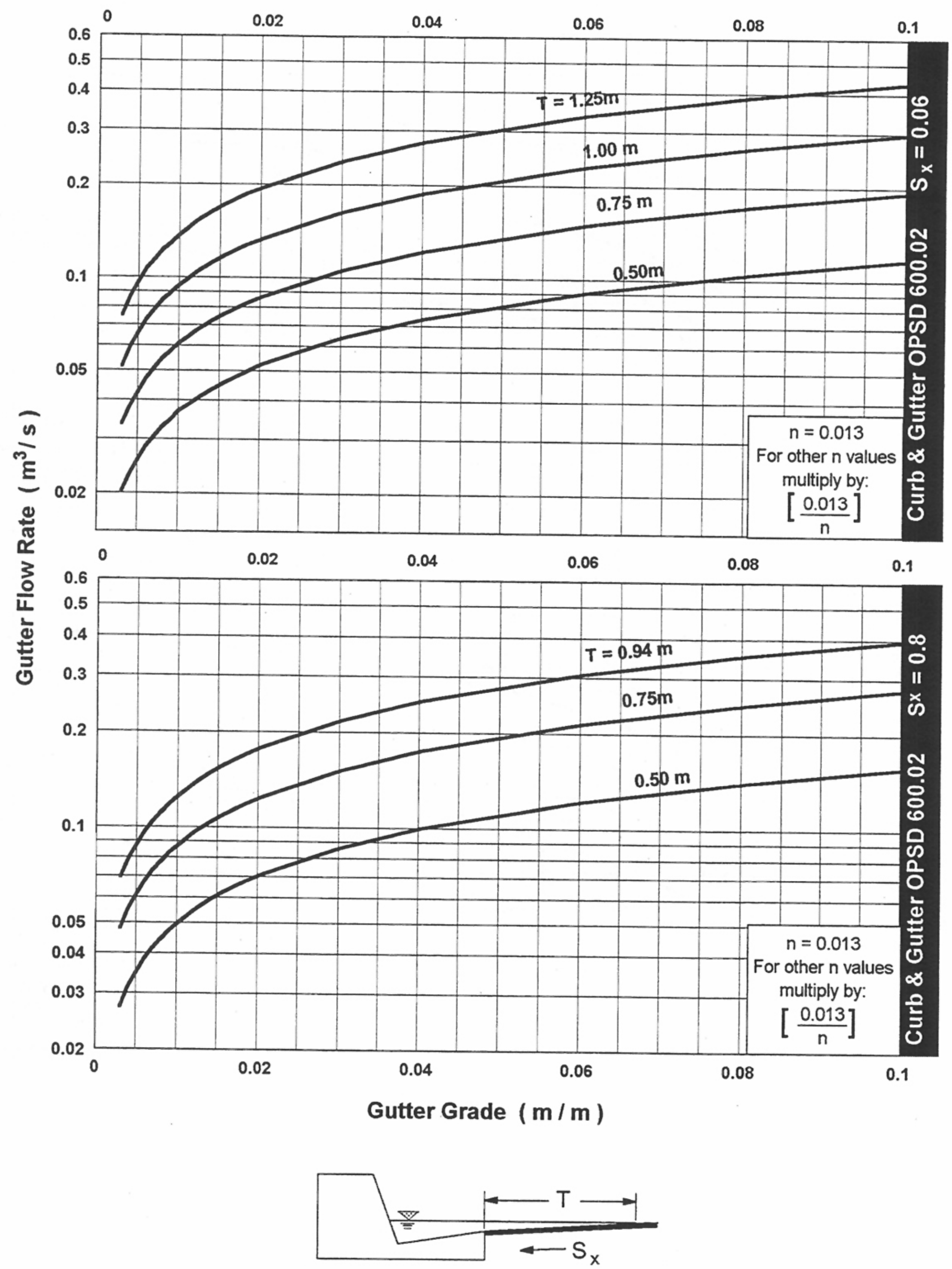
Design Chart 4.05: Gutter Flow Rate - Curb & Gutter OPSD 600.01



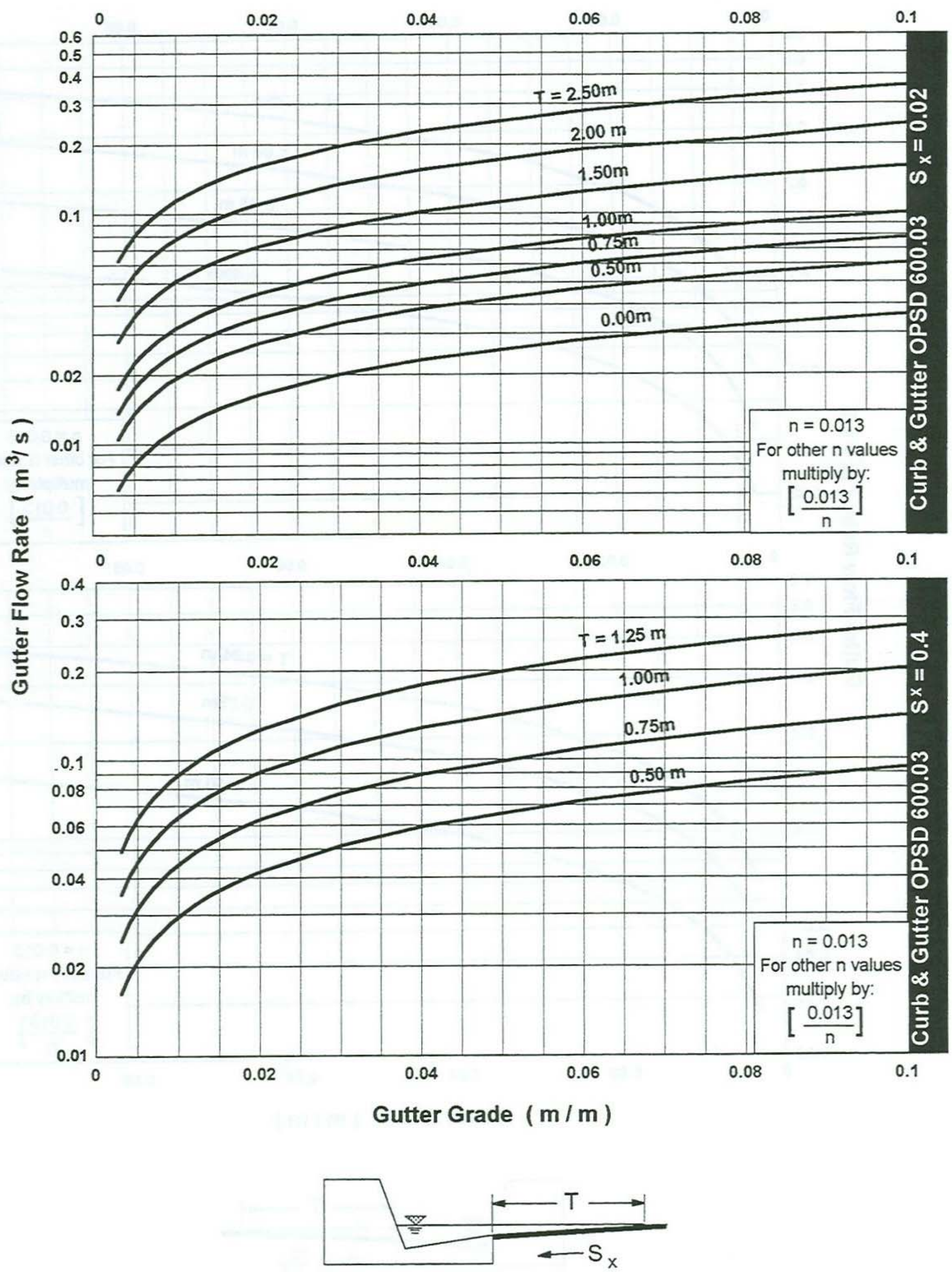
Design Chart 4.06: Gutter Flow Rate - Curb & Gutter OPSD 600.02



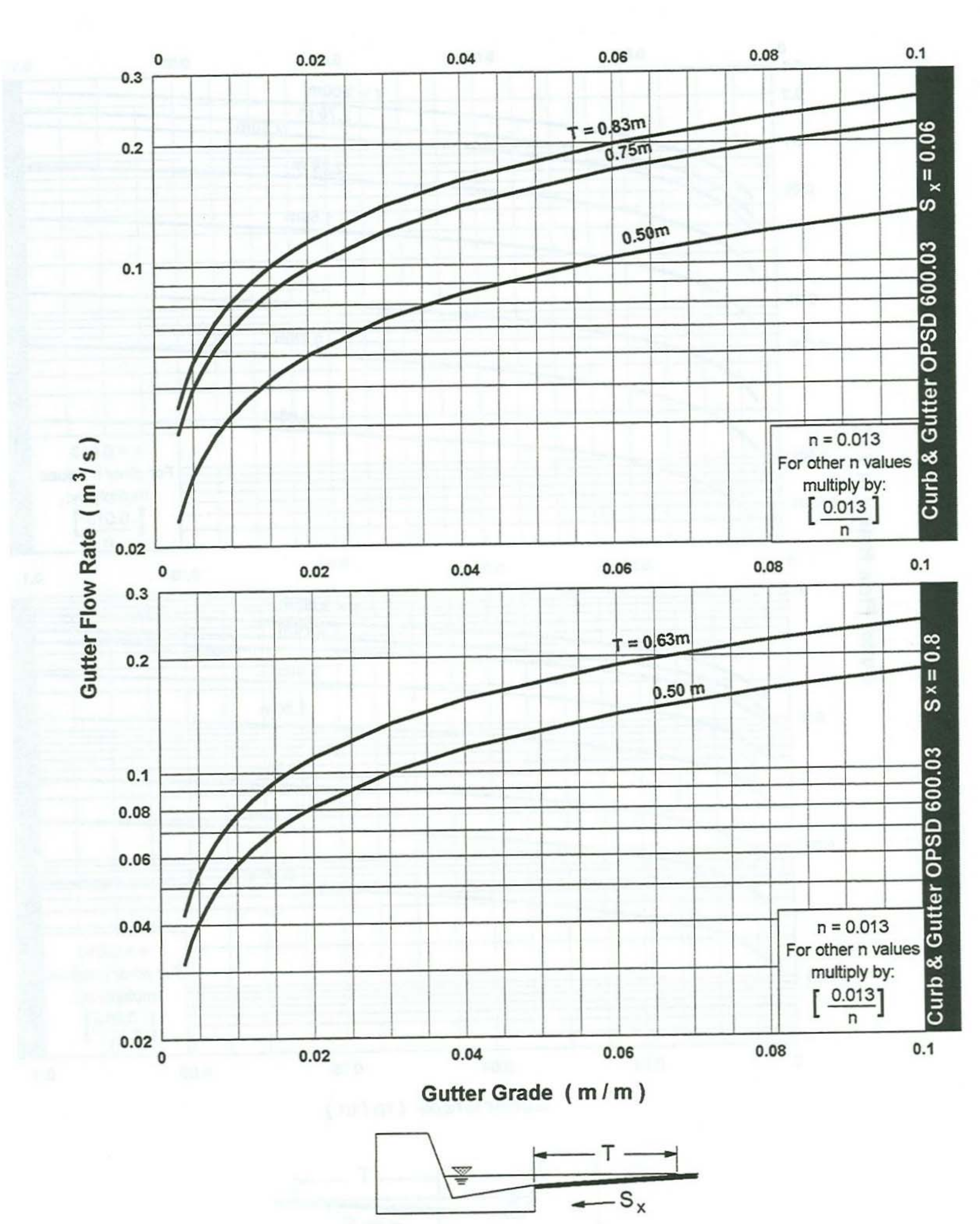
Design Chart 4.07: Gutter Flow Rate - Curb & Gutter OPSD 600.02



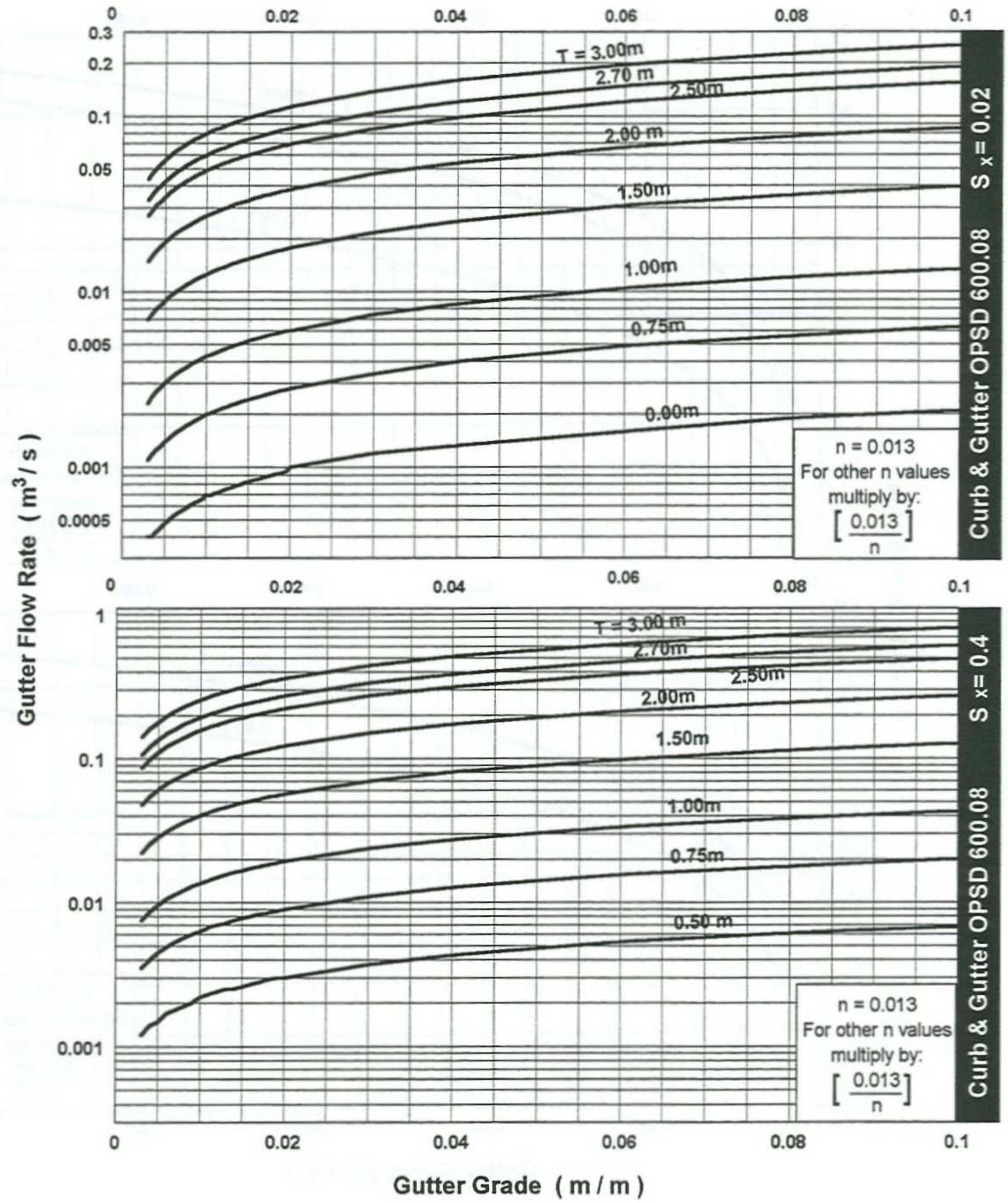
Design Chart 4.08: Gutter Flow Rate - Curb & Gutter OPSD 600.03

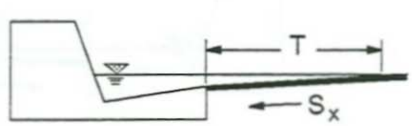


Design Chart 4.09: Gutter Flow Rate - Curb & Gutter OPSD 600.03

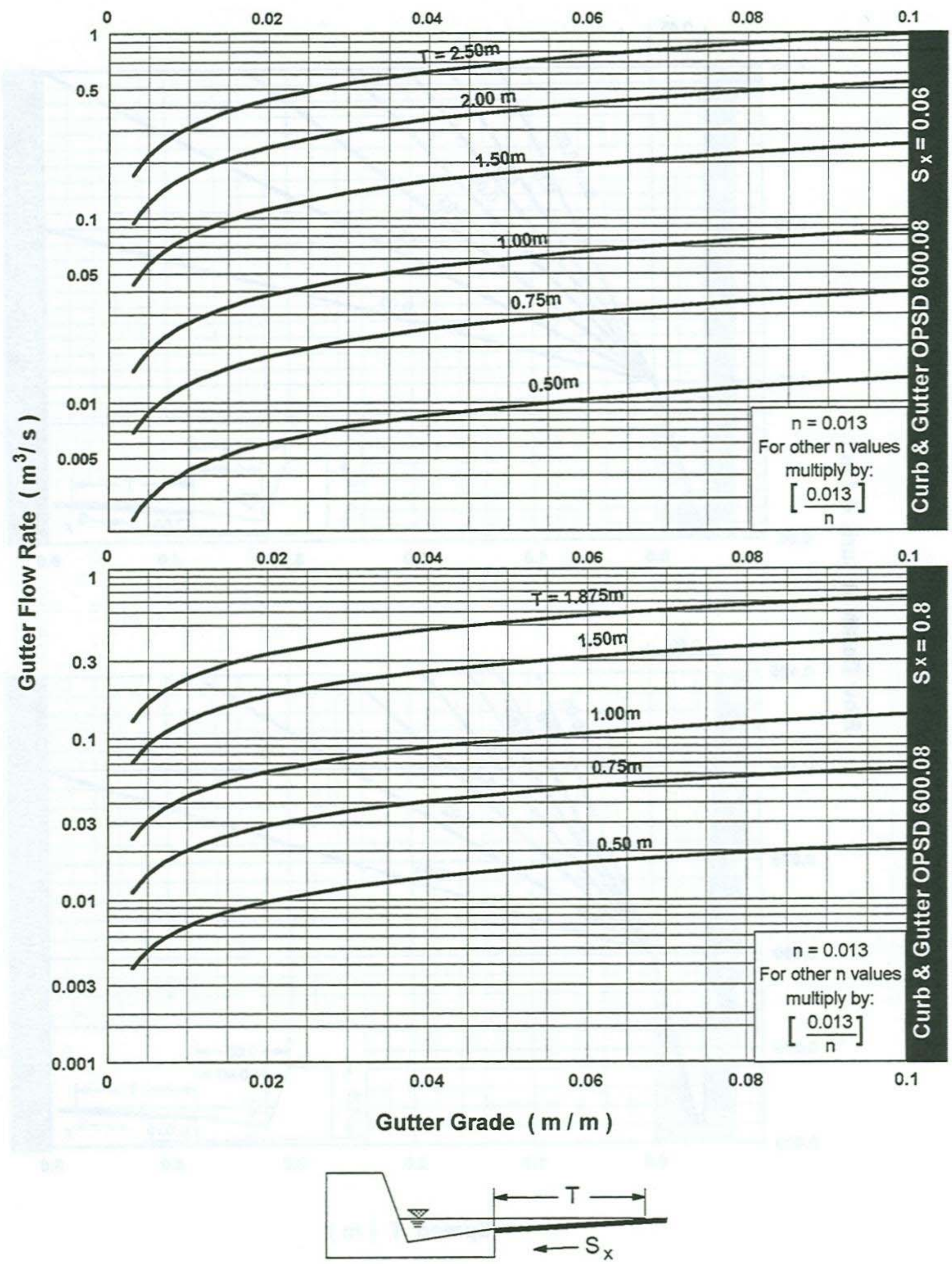


Design Chart 4.10: Gutter Flow Rate - Curb & Gutter OPSD 600.08

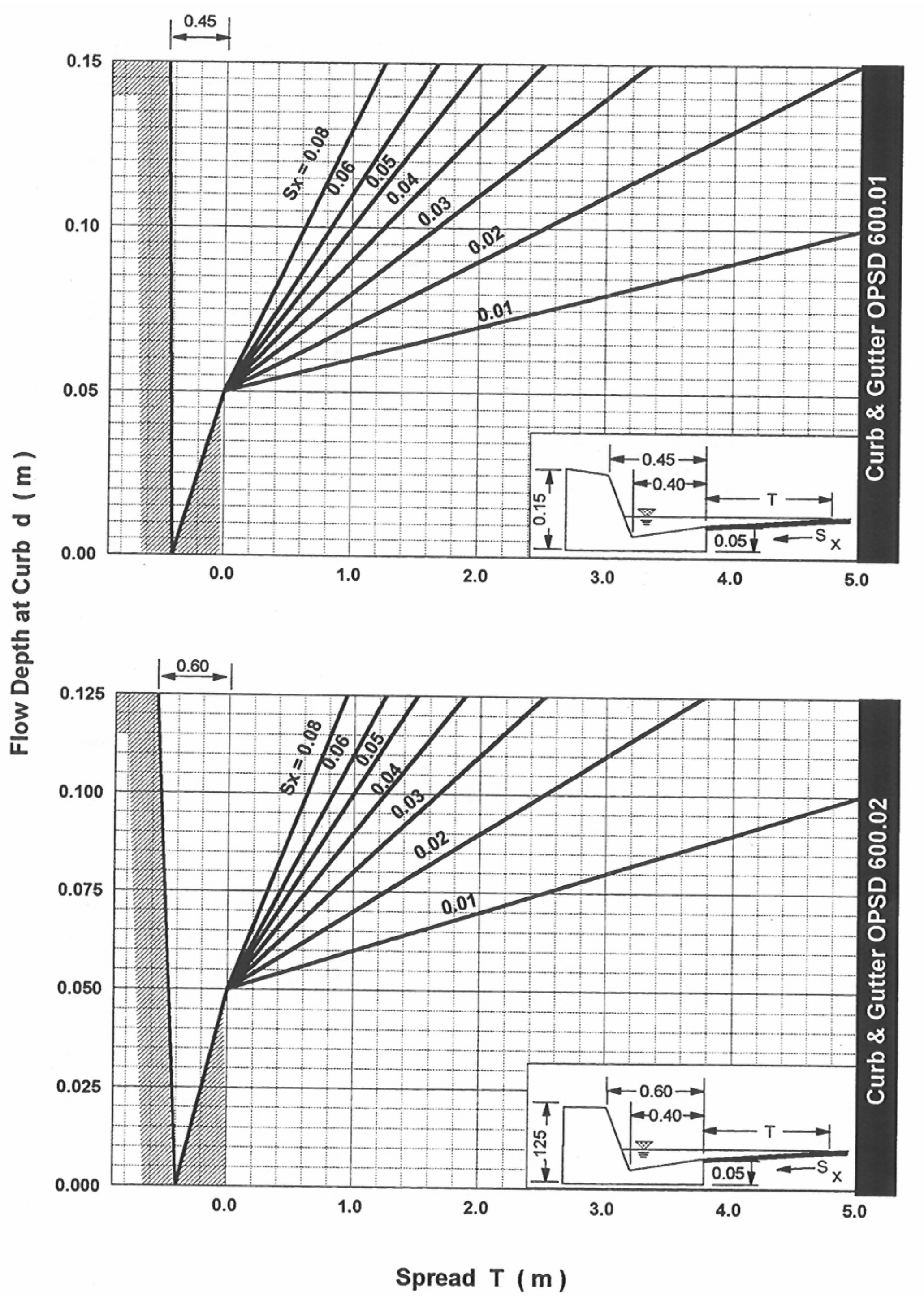




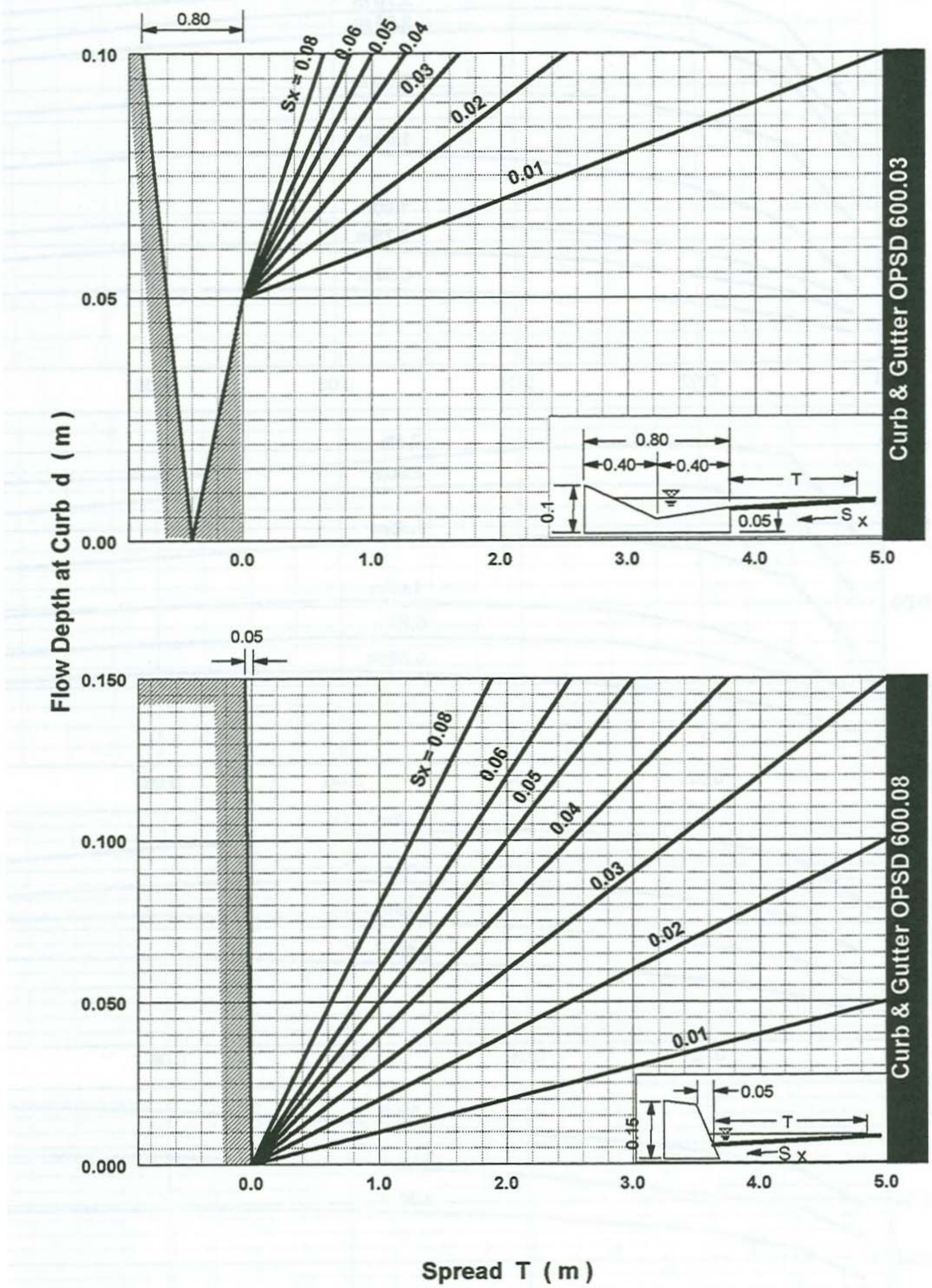
Design Chart 4.11: Gutter Flow Rate - Curb & Gutter OPSD 600.08



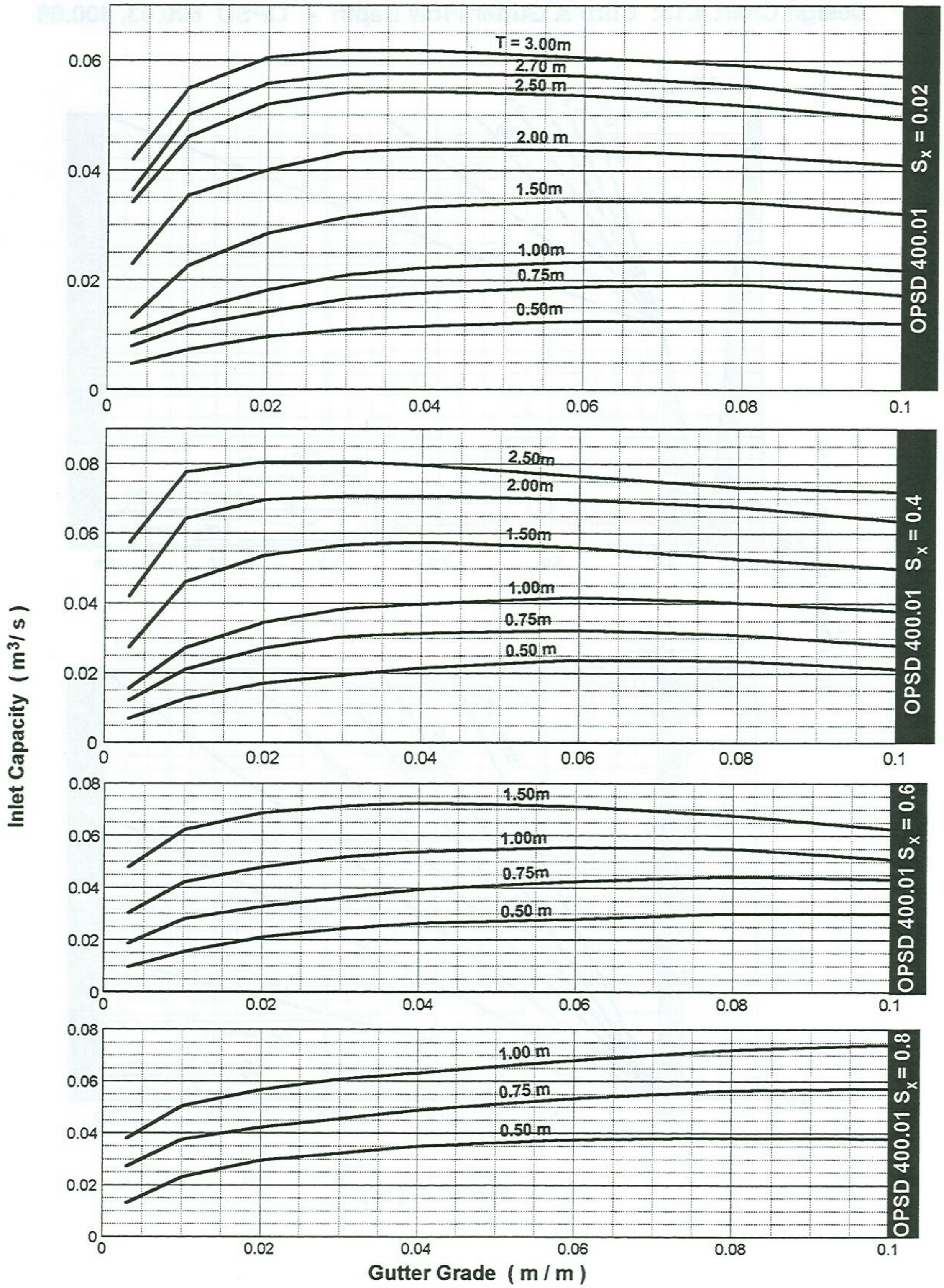
Design Chart 4.12: Curb & Gutter Flow Depth -OPSD 600.02



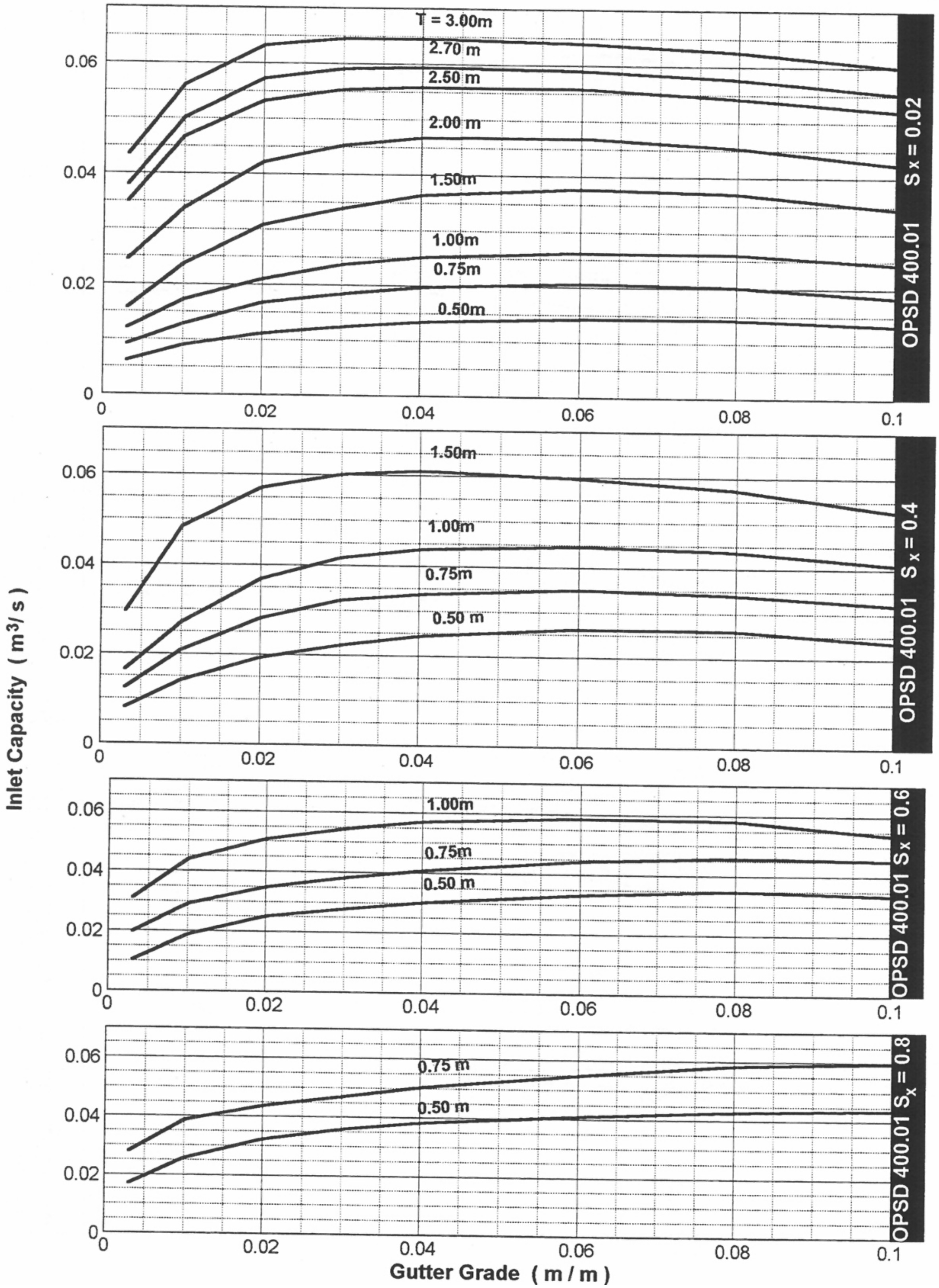
Design Chart 4.13: Curb & Gutter Flow Depth - OPSD 600.03, 600.08



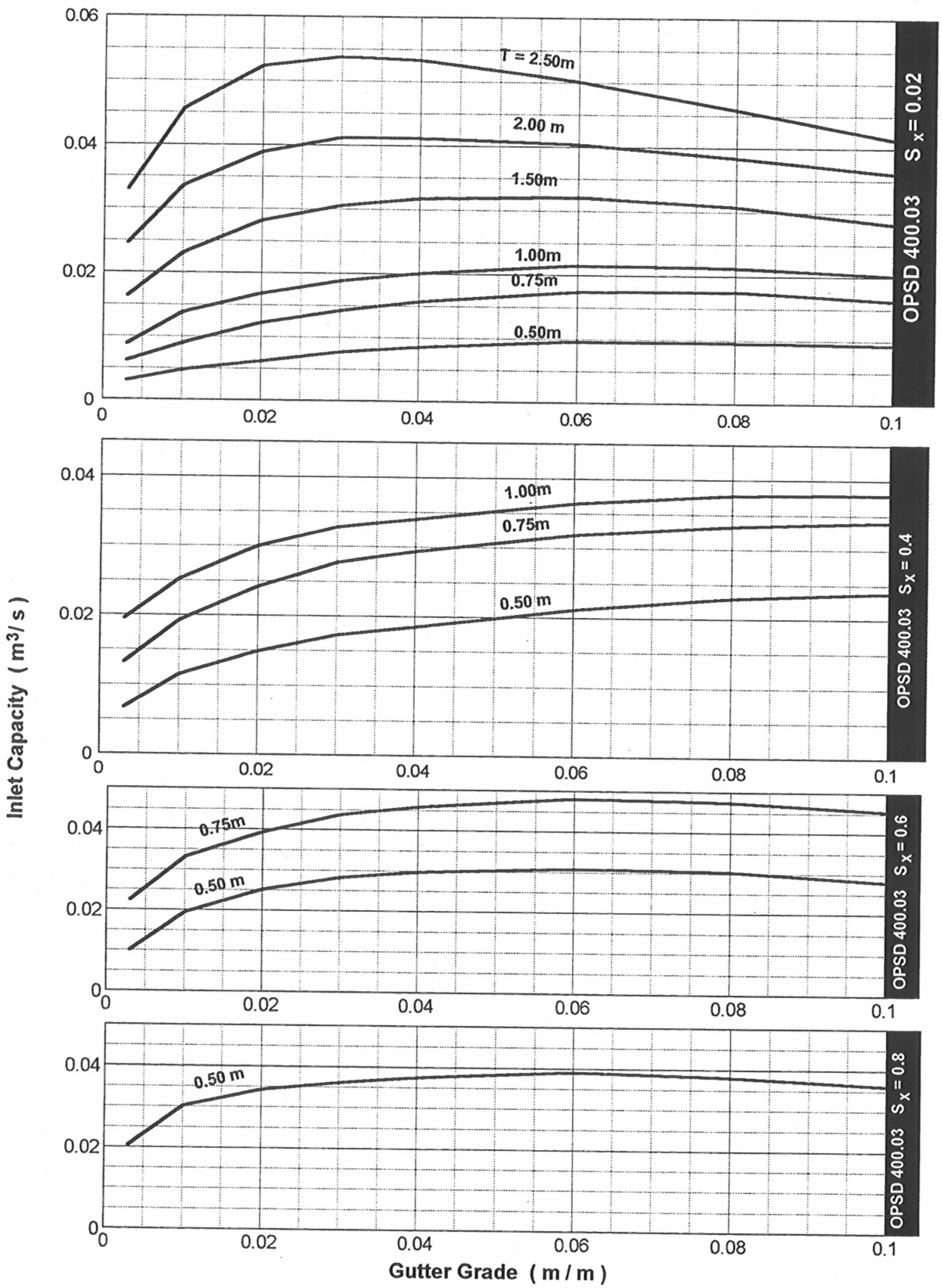
Design Chart 4.14: Inlet Capacity OPSD 400.01 (C & G OPSD 600.01)



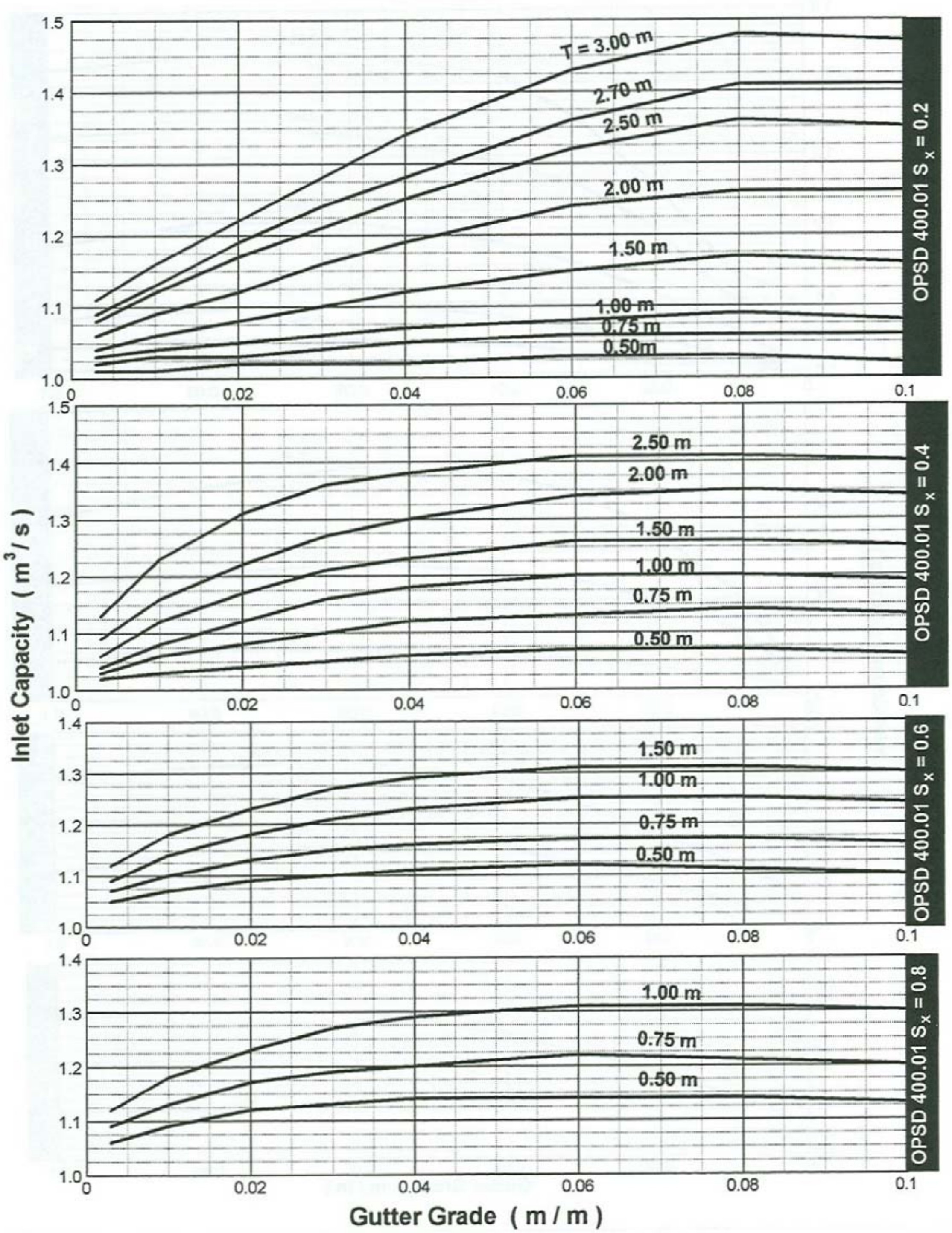
Design Chart 4.15: Inlet Capacity OPSD 400.01 (C & G OPSD 600.02)



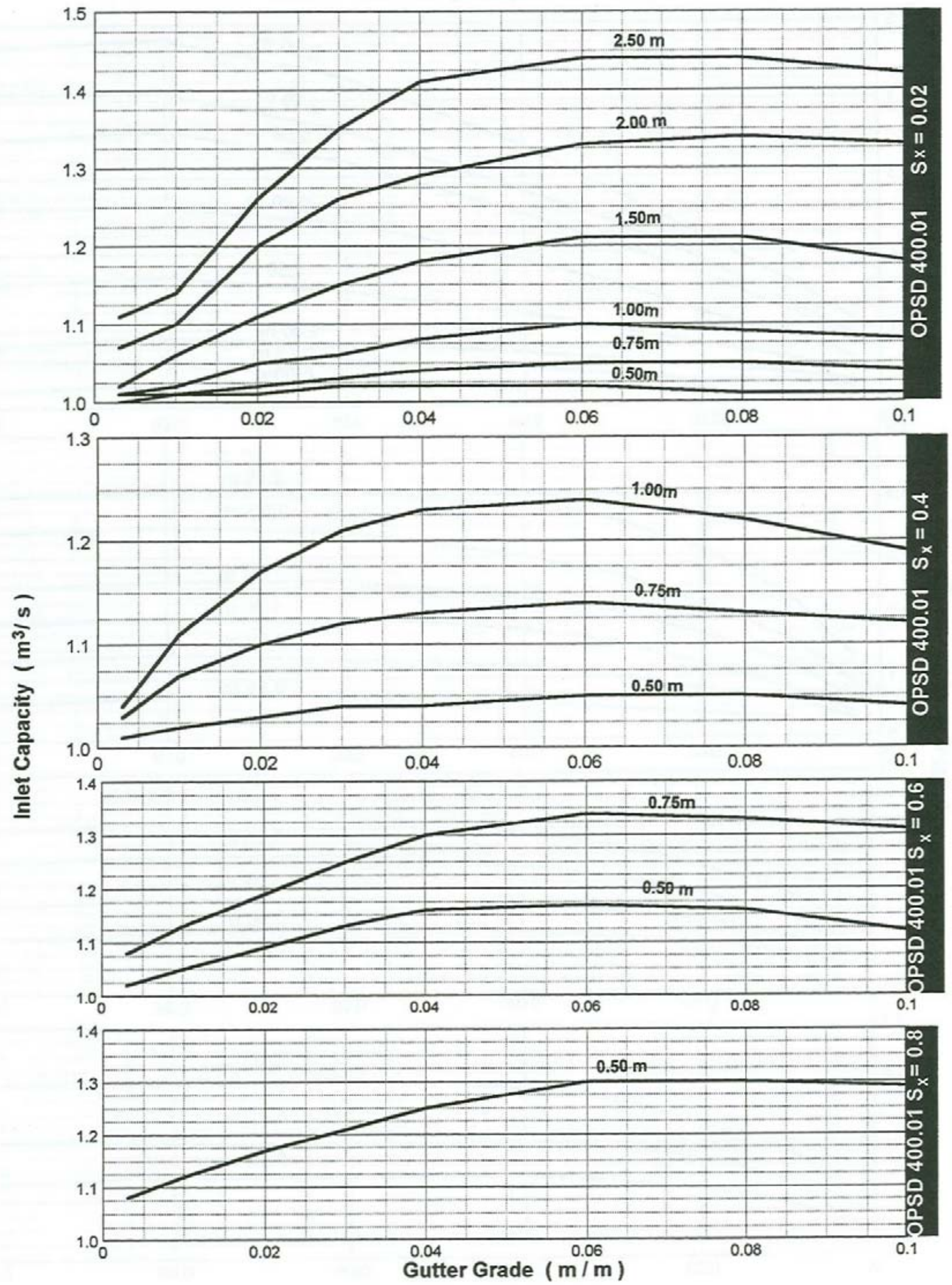
Design Chart 4.16: Inlet Capacity OPSD 400.03 (C & G OPSD 600.03)



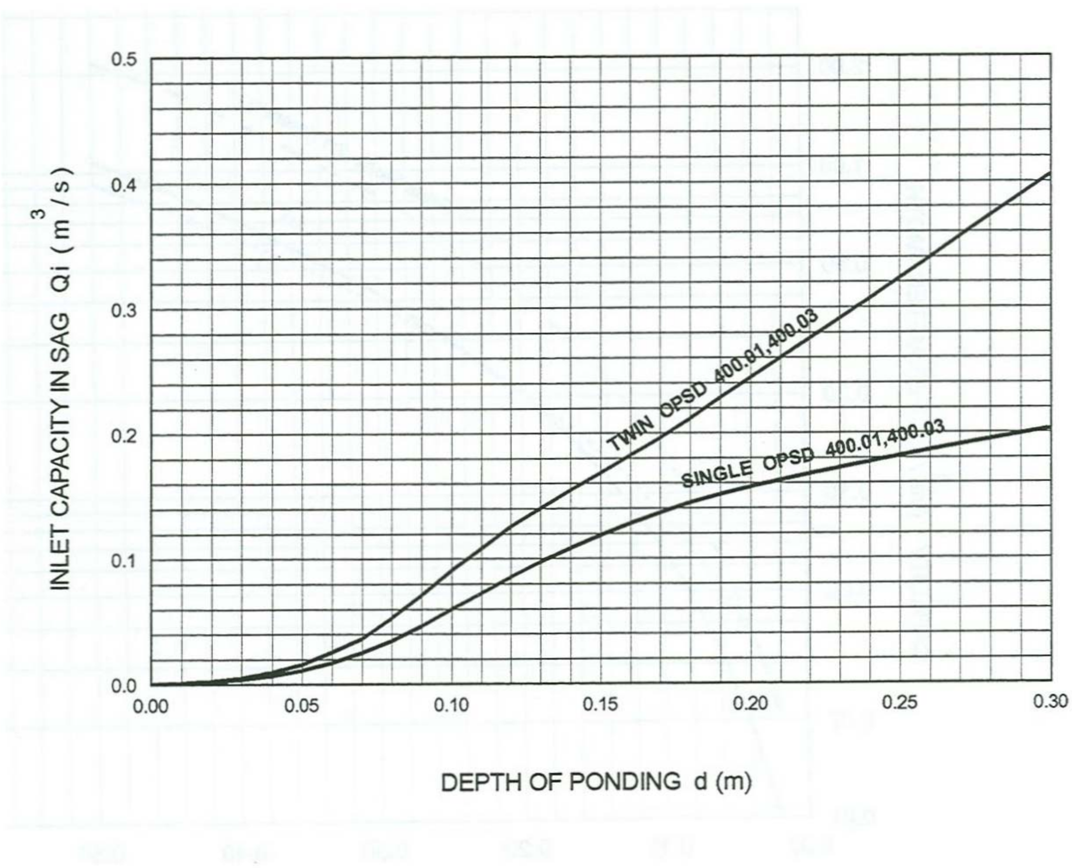
Design Chart 4.17: Twin Inlet Capacity OPSD 400.01



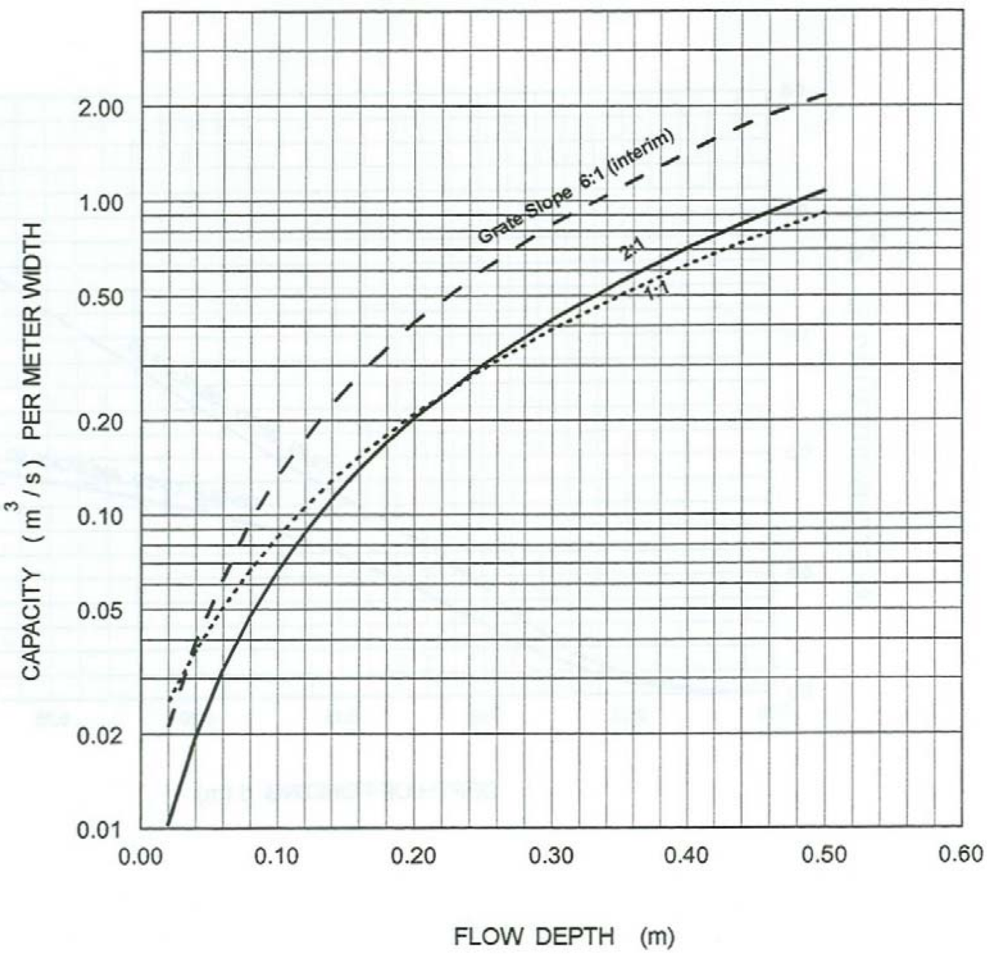
Design Chart 4.18: Twin Inlet Capacity OPSD 400.03



Design Chart 4.19: Inlet Capacity at Road Sag



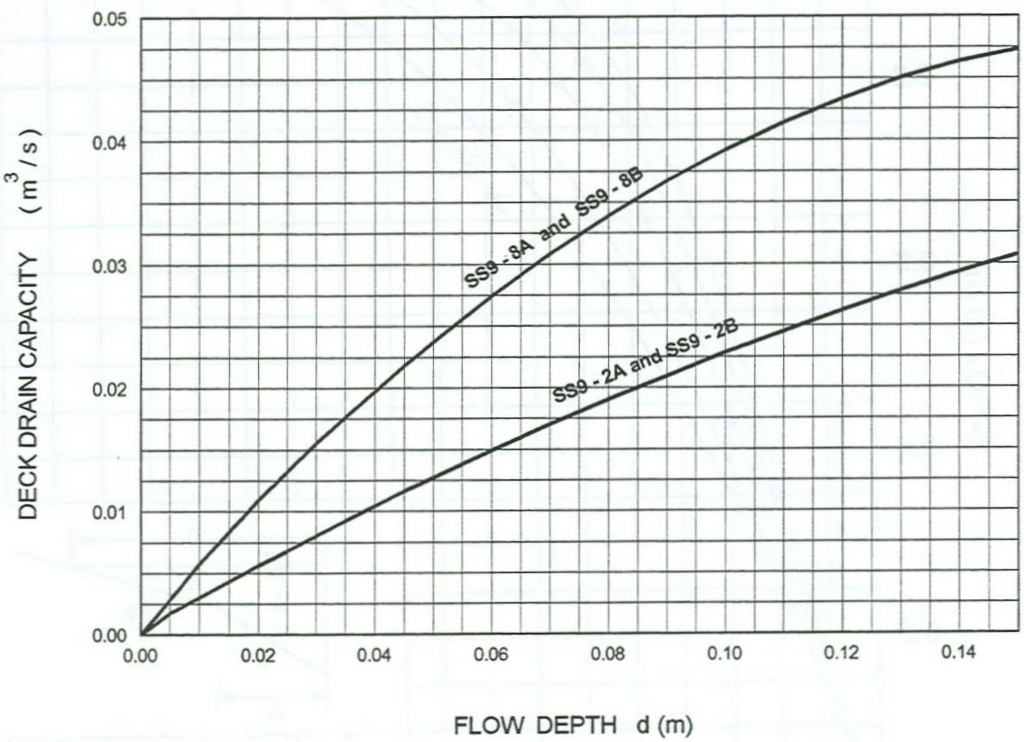
Design Chart 4.20: Ditch Inlet Capacity



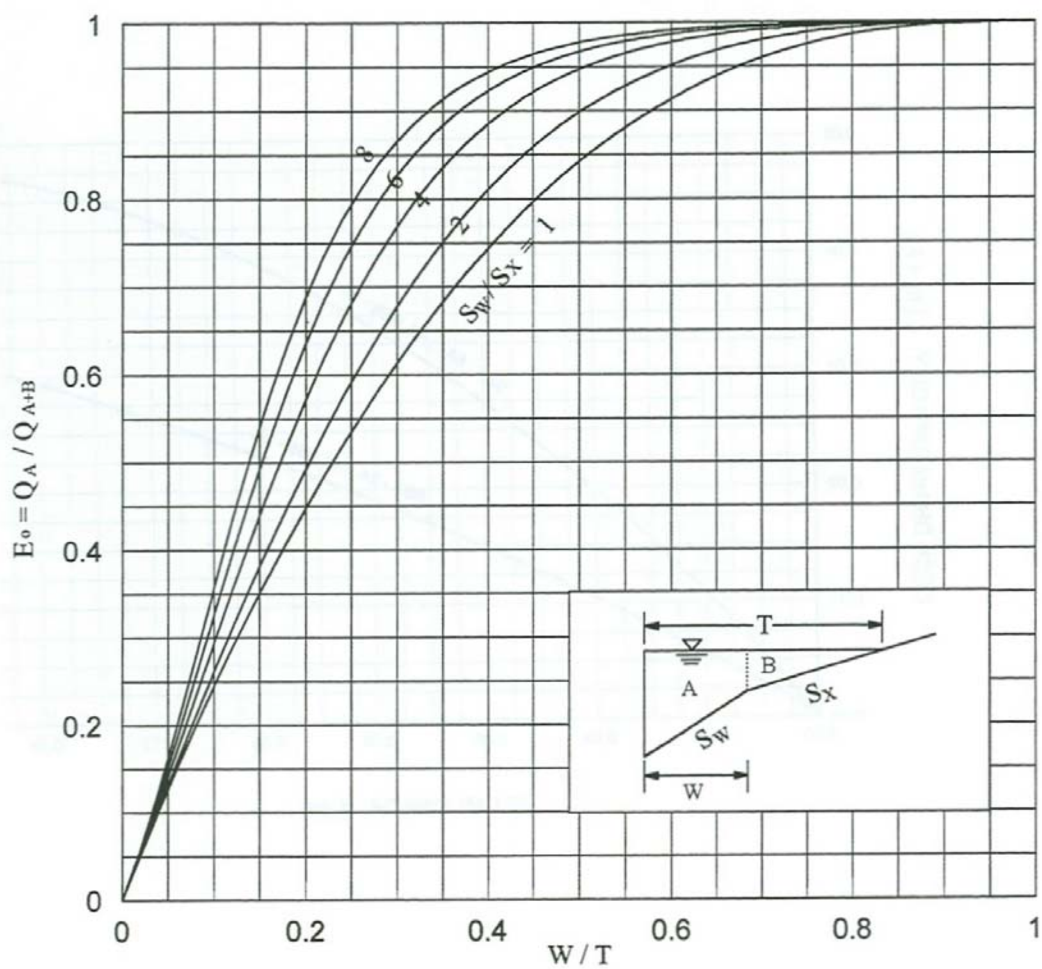
Notes:

1. Curves apply to grate Type 403.01, but may be used for straight - bar inlets without significant loss of accuracy.
2. Capacities given by curves are for unobstructed grates only. For design use working capacity ≯ 0.5 x unobstructed capacity.
3. Capacities of grates operating in high velocity flows are less than indicated.

Design Chart 4.21: Bridge Inlet Capacity

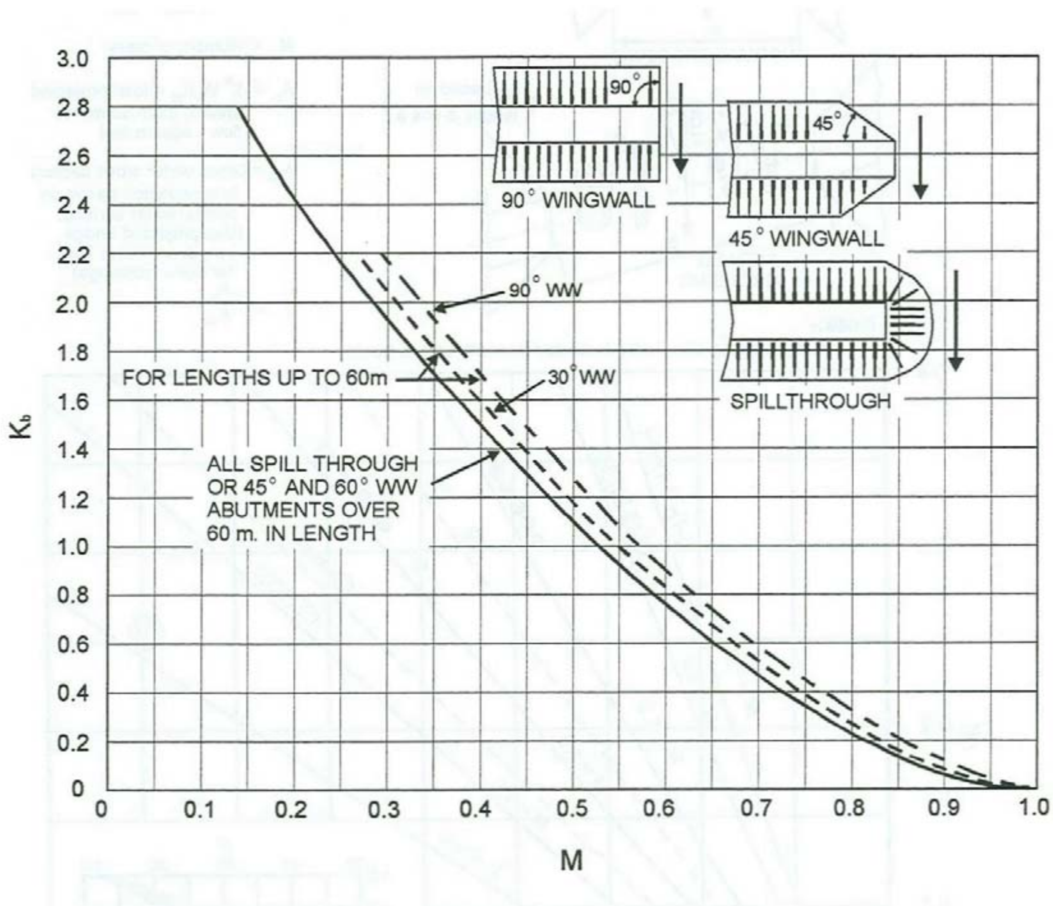


Design Chart 4.22: Ratio of Frontal Flow to Total Gutter Flow



Source: Hec-12

Design Chart 5.01: Base Coefficient - Bridge Backwater

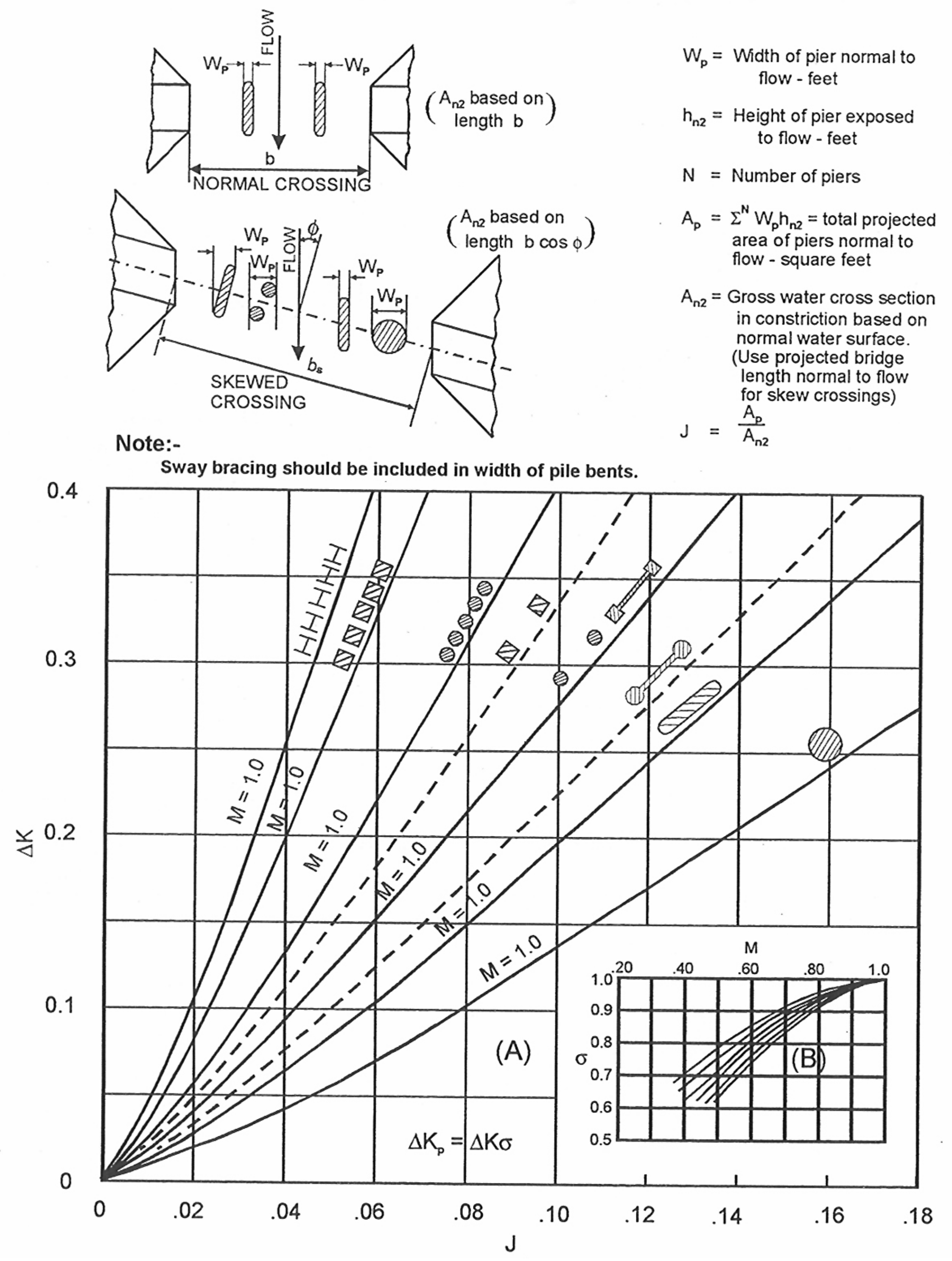


Conveyance Ratio M

$$M = \frac{\text{Unimpeded Flow through bridge opening, } m^3/s}{\text{Total flow from opening and flood plain, } m^3/s}$$

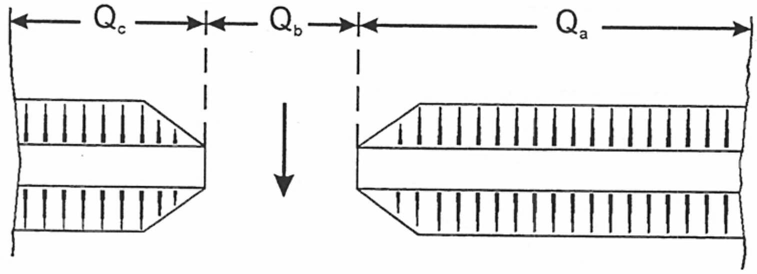
Source: Bradley (1978)

Design Chart 5.02: Pier Coefficient - Bridge Backwater



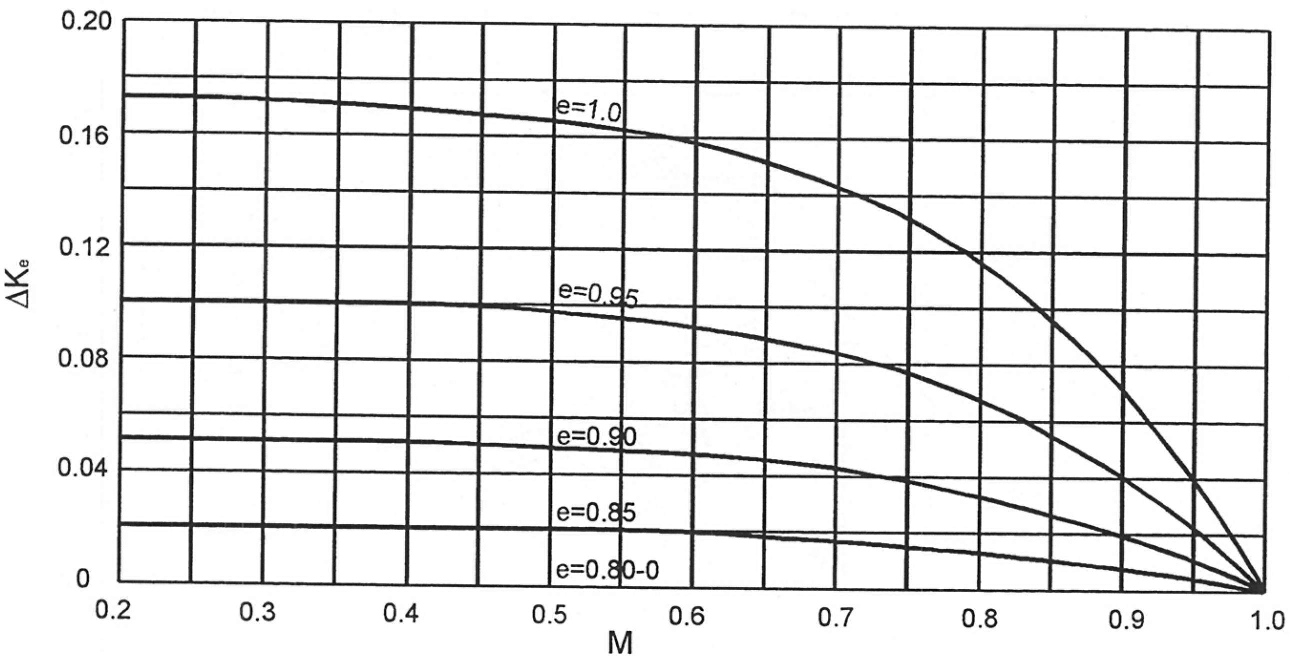
Source: Bradley (1978)

Design Chart 5.03: Eccentricity Coefficient - Bridge Backwater



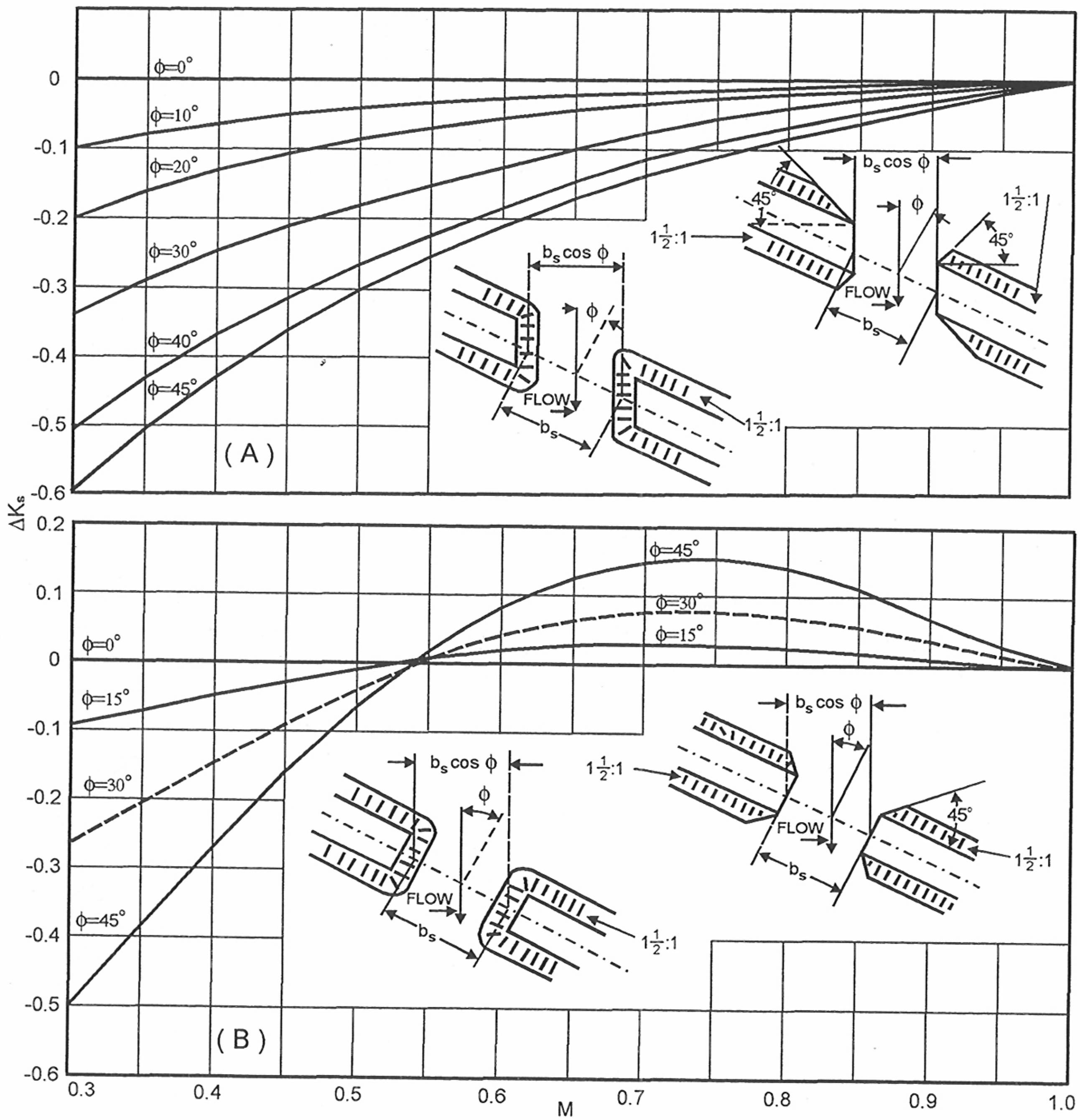
$$e = \left(1 - \frac{Q_c}{Q_a}\right) \quad \text{where } Q_c < Q_a \quad \text{or}$$

$$e = \left(1 - \frac{Q_a}{Q_c}\right) \quad \text{where } Q_a < Q_c$$



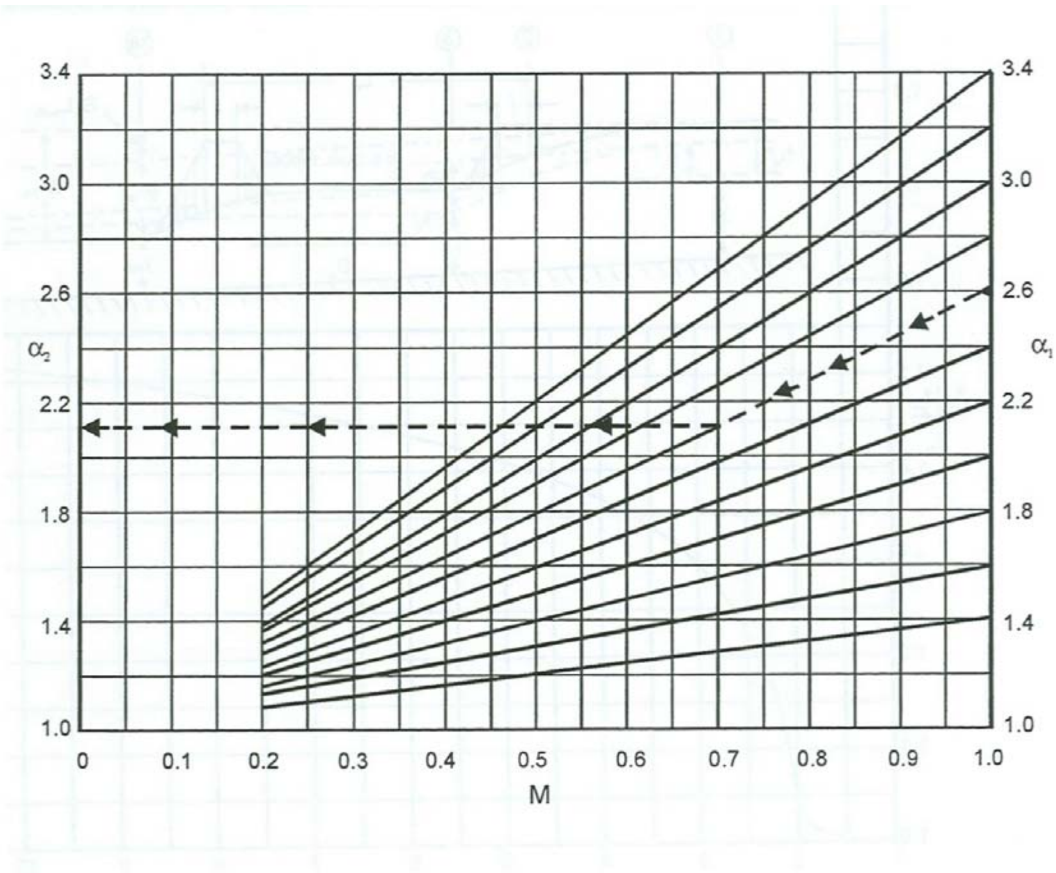
Source: Bradley (1978)

Design Chart 5.04: Skew Coefficient - Bridge Backwater



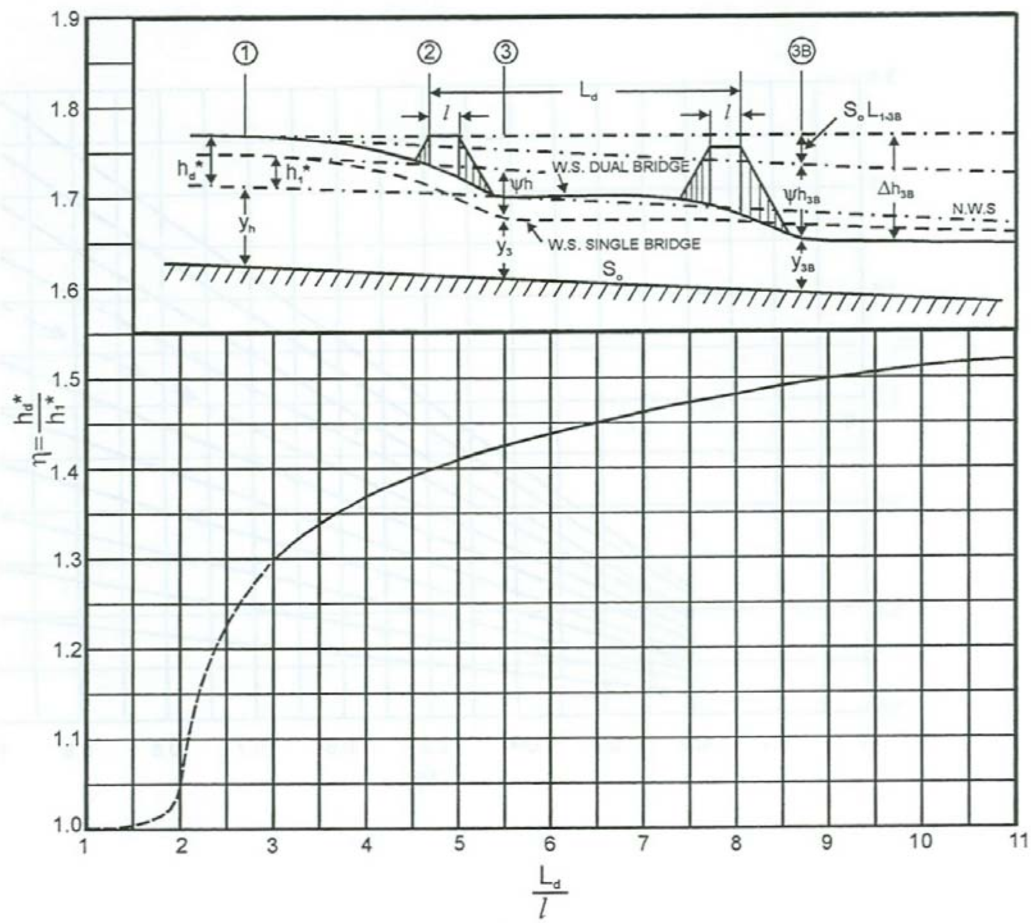
Source: Bradley (1978)

Design Chart 5.05: Velocity Head Coefficient - Bridge Backwater



Source: Bradley (1978)

Design Chart 5.06: Backwater Adjustment for Parallel Bridges



Source: Bradley (1978)

Design Chart 5.07: Competent Velocity Table - Cohesive Soils

Depth of Flow (m)	Soil Scourability **		
	High (m/s)	Medium (m/s)	Low (m/s)
1.0	0.5	0.9	1.6
1.5	0.6	1.0	1.8
3.0	0.6	1.2	2.0
6.0	0.7	1.3	2.3
15.0	0.8	1.5	2.6

* Competent velocities should be based on local experience whenever possible, taking into account saturation & weathering.

** It is not considered advisable to relate the tabulated values to soil property indices because of the strong effect of satuaration and weathering on the scourability of soils. However the following tentative relationship to soil consistency is offered as a rough guide.

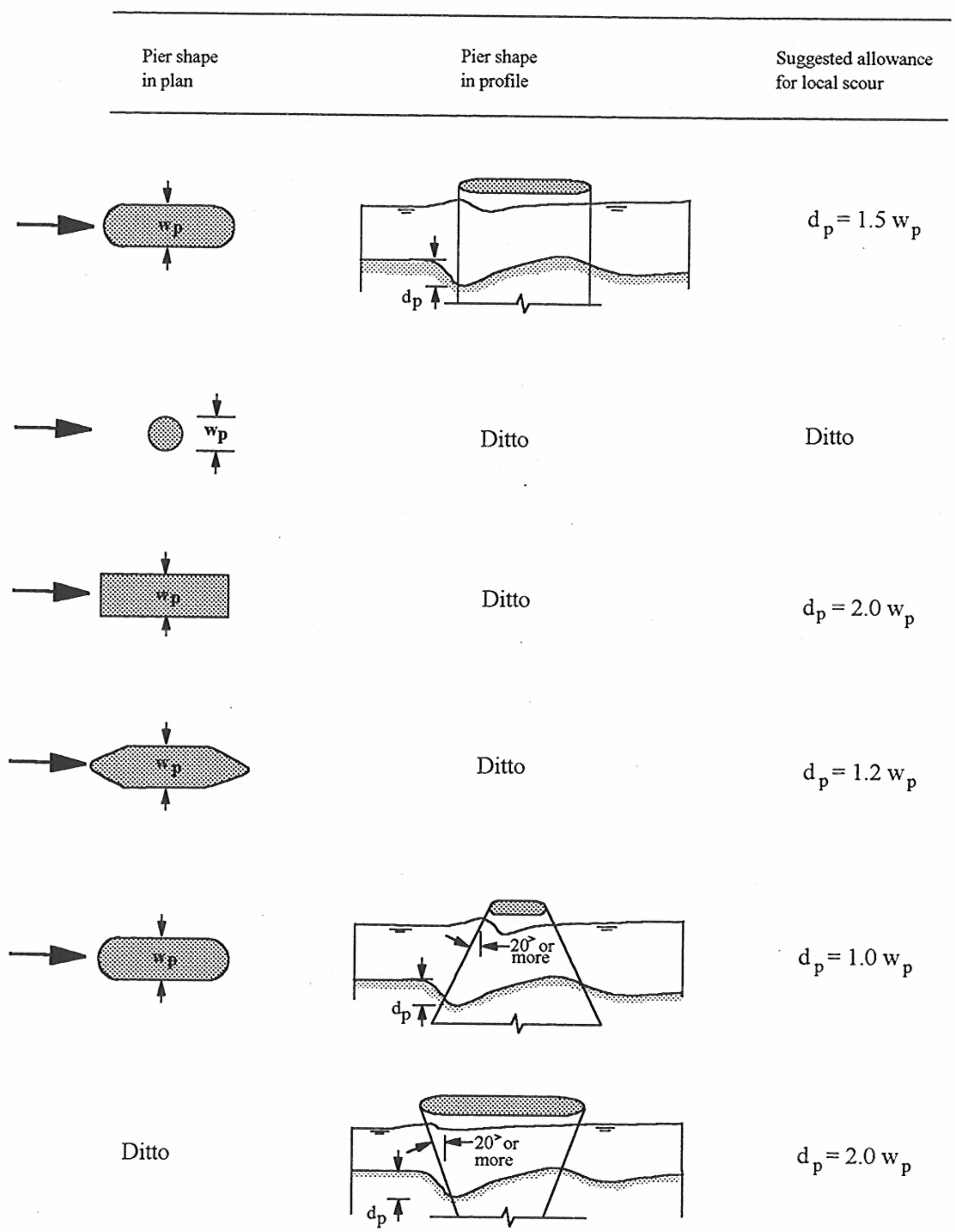
High scourablility .... very soft to soft clays
Medium scourability .... firm to stiff clays
Low scourability.................... very stiff to hard calys, some glacial tills.

Soil consistency can be judged by the following field tests applied with the soil at or near its natural water content.

Very soft:	easily penetrated several centimeters by fist
Soft:	easily penetrated several centimeters by thumb
Firm:	moderate effort required to penetrate several centimeters by thumb
Stiff:	readily indented , but penetrated only be great effort by thumb
Very stiff:	redily indented by thumbnail
Hard:	indented with difficulty by thumbnail

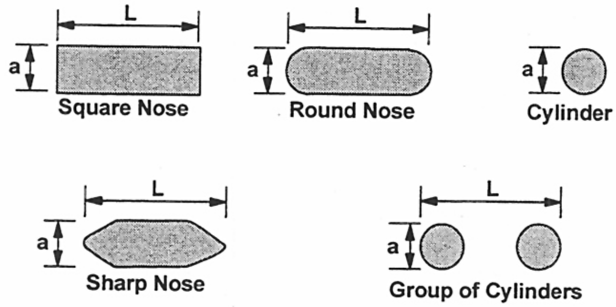
Source: Neill (1993)

Design Chart 5.08: Estimating Local Pier Scour



Source: Neill (1973)

Design Chart 5.09: Pier Shape Correction Factors (K_1 and K_2)



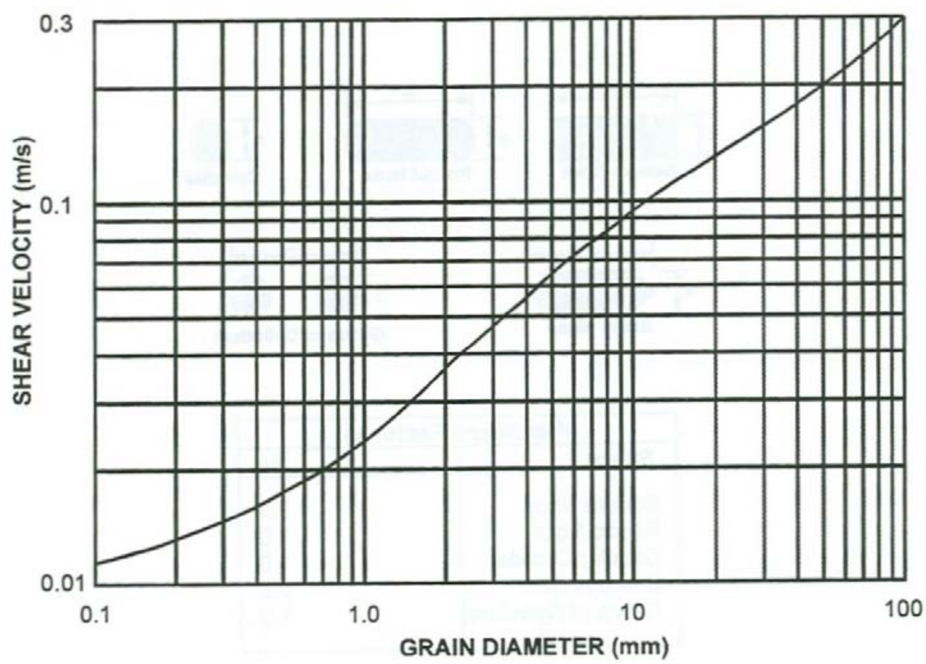
Pier Shape Factor k_1	
Shape	**k_1**
Square Nose	1.1
Round Nose	1.0
Circular Cylinder	1.0
Sharp Nose	0.9
Group of Cylinders	1.0

Correction Factor k_2			
Angle	**k_2**		
	L/a=4	**L/a=8**	**L/a=12**
0	1.0	1.0	1.0
15	1.5	2.0	2.5
30	2.0	2.5	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0
Angle = skew angle of flow			

Note: The correction factor k_1 for pier nose shape should be determined using the table for angle of attack up to 5 degrees. For greater angles, pier nose shape loses its affect and k_1 should be considered as 1.0.

Source: U.S. FHWA - Hydraulic Circular No. 18 (1991)

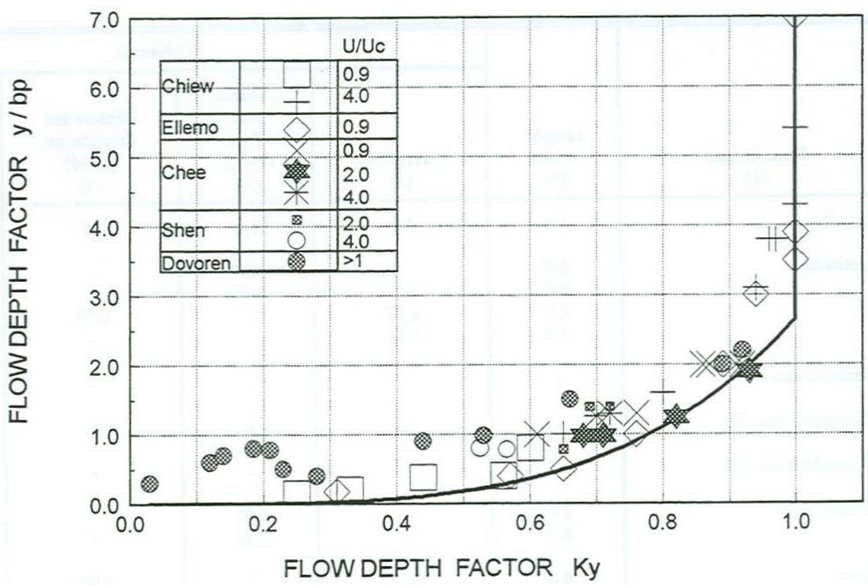
Design Chart 5.10: Shield's Chart



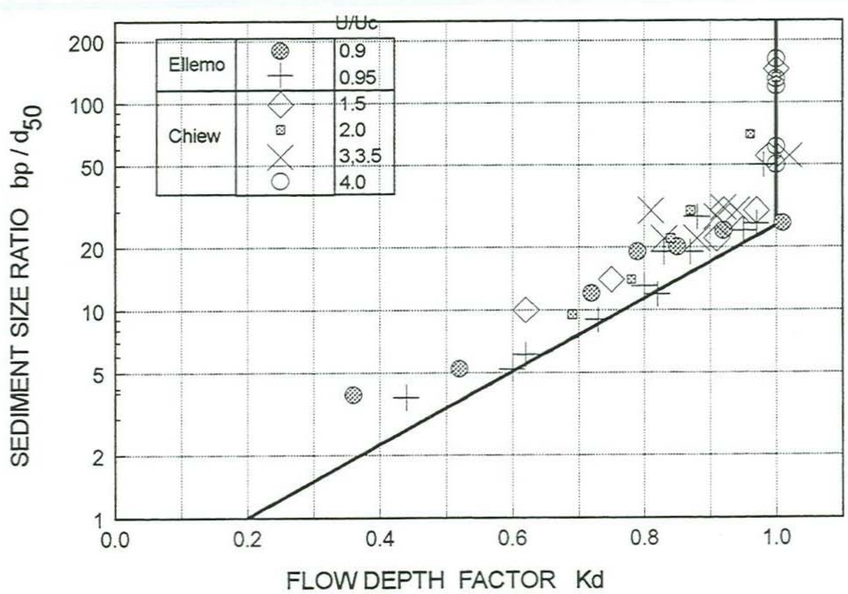
Shields Chart for Threshold Condition of Uniform Sediments in Water

Source: Melville & Sutherland (1988)

Design Chart 5.11: Flow Depth Factors (k_y)



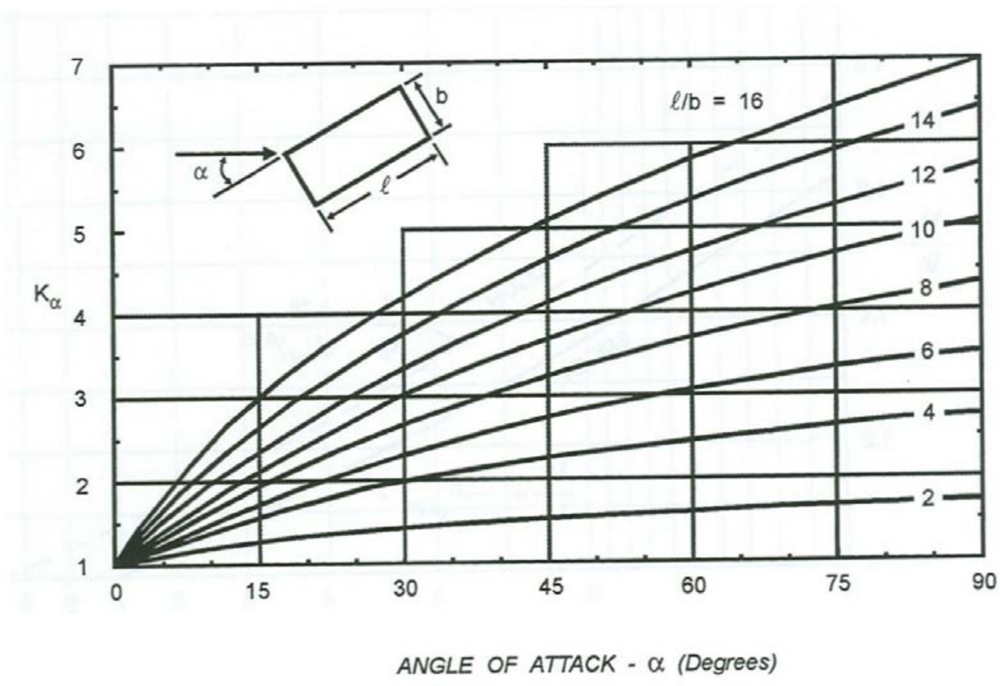
Design Chart 5.12: Sediment Size Factors (k_d)



Design Chart 5.13: Pier Shape Correction

Shape in plan (1)	Length/ width (2)	Reference			
		Tison (1940) (3)	Laurens and Toch (1956) (4)	Chabert and Engeldinger (1956) (5)	Venkatadri (1965) (6)
Circular	1.0	1.0	1.0	1.0	1.0
Lenticular	2.0	-	0.97	-	-
	3.0	-	0.76	-	-
	4.0	0.67	-	0.73	-
	7.0	0.41	-	-	-
Parabolic nose	-	-	-	-	0.56
Triangular nose, 60E	-	-	-	-	0.75
Triangular nose, 90E	-	-	-	-	1.25
Elliptic	2.0	-	0.91	-	-
	3.0	-	0.83	-	-
Ogival	4.0	0.86	-	0.92	-
Joukowski	4.0	-	-	0.86	-
	4.1	0.76	-	-	-
Rectangular	2.0	-	1.11	-	-
	4.0	1.40	-	1.11	-
	6.0	-	1.11	-	-

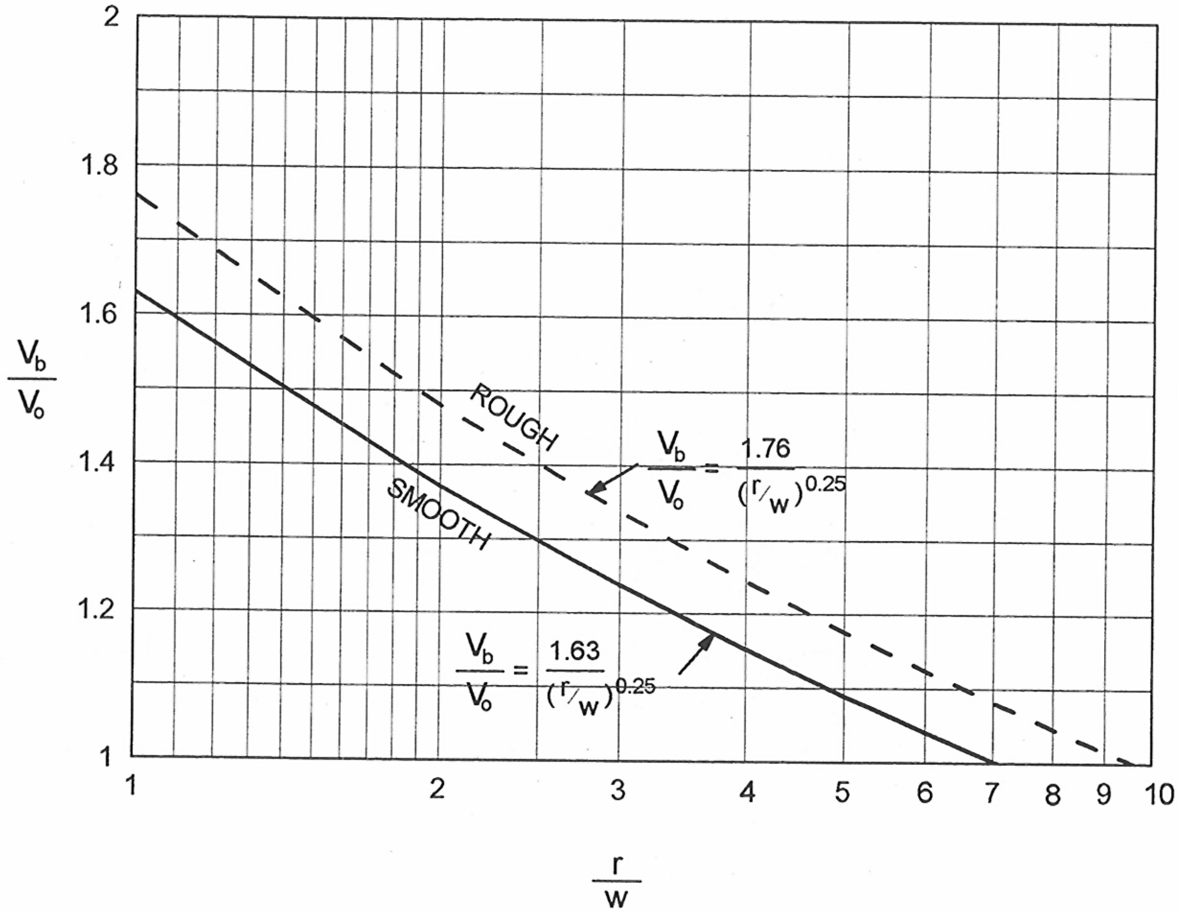
Design Chart 5.14: Pier Alignment Factors



Alignment Factor K_α for Piers Not Aligned with Flow

Source: Melville and Sutherland (1988)

Design Chart 5.15: Flow Velocity - Channel Curvature Chart

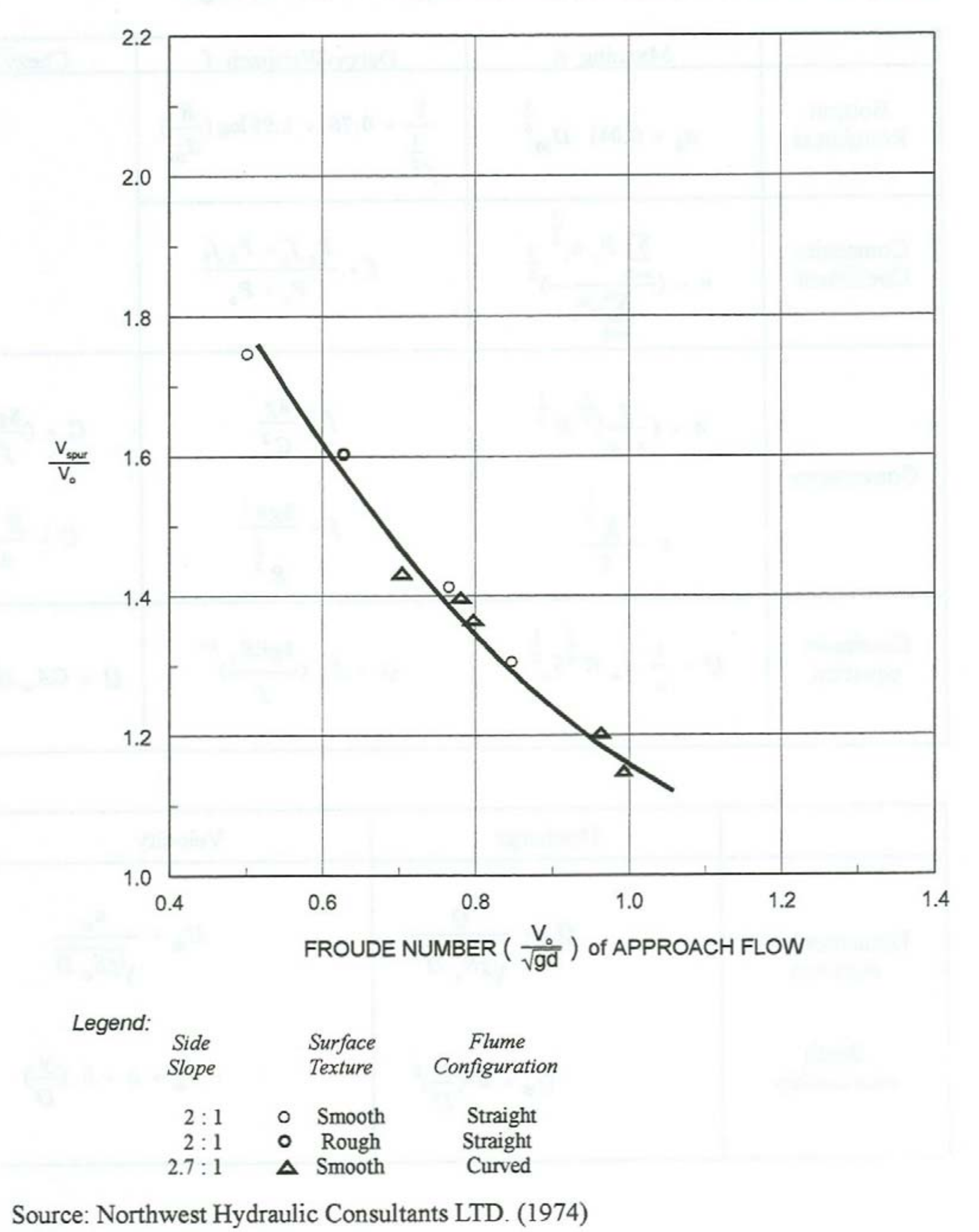


Legend:

V_b - Maximum velocity in bend
V_o - Average approach velocity
r - Centerline radius of bend
w - Average channel width

Source: Northwest Hydraulic Consultants Ltd (1974)

Design Chart 5.16: Local Acceleration Chart - Groynes



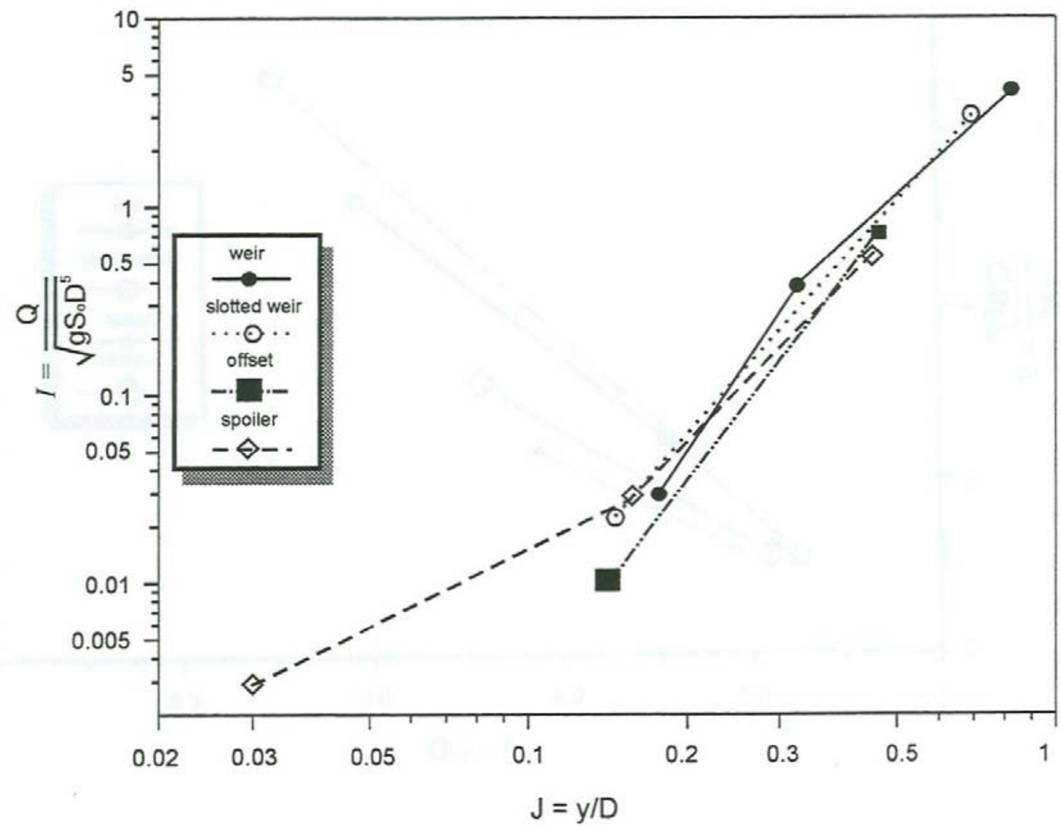
Source: Northwest Hydraulic Consultants (1974)

Design Chart 5.17: Hydraulic Relationships for Fish Passage

	Manning n	Darcy-Weisbach f	Chezy C
Bottom Roughness	$n_b = 0.041\ D_{50}^{\frac{1}{6}}$	$\frac{1}{f^{\frac{1}{2}}} = 0.76 + 1.98 \log(\frac{R}{d_{50}})$	
Composite Coefficient	$n = (\frac{\sum_{i=c,b} P_i n_i^{\frac{3}{2}}}{\sum_{i=c,b} P_i})^{\frac{2}{3}}$	$f = \frac{P_c f_c + P_b f_b}{P_c + P_b}$	
Conversions	$n = (\frac{f}{8\ g})^{\frac{1}{2}} R^{\frac{1}{6}}$ $n = \frac{R^{\frac{1}{6}}}{C}$	$f = \frac{8g}{C^2}$ $f = \frac{8gn^2}{R^{\frac{1}{3}}}$	$C = (\frac{8g}{f})^{\frac{1}{2}}$ $C = \frac{R^{\frac{1}{6}}}{n}$
Continuity equation	$Q = \frac{1}{n} A_w R^{\frac{2}{3}} S_o^{\frac{1}{2}}$	$Q = A_w (\frac{8gRS_o}{f})^{1/2}$	$Q = CA_w (RS)^{1/2}$

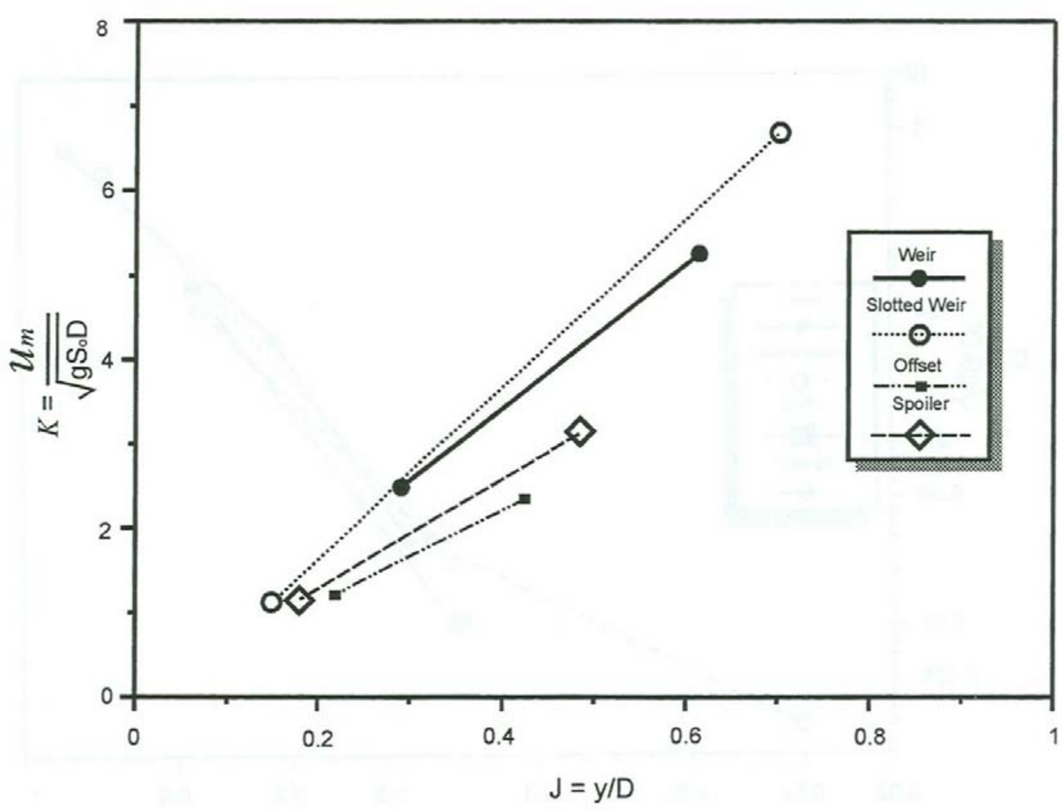
	Discharge	Velocity
Dimensionless equation	$Q_* = \frac{Q}{\sqrt{gS_o D^5}}$	$U_* = \frac{u_m}{\sqrt{gS_o D}}$
Depth relationship	$Q_* = a\ (\frac{y}{D})^b$	$U_* = a + b\ (\frac{y}{D})$

Design Chart 5.18: Hydraulic Relationship of "*I*"



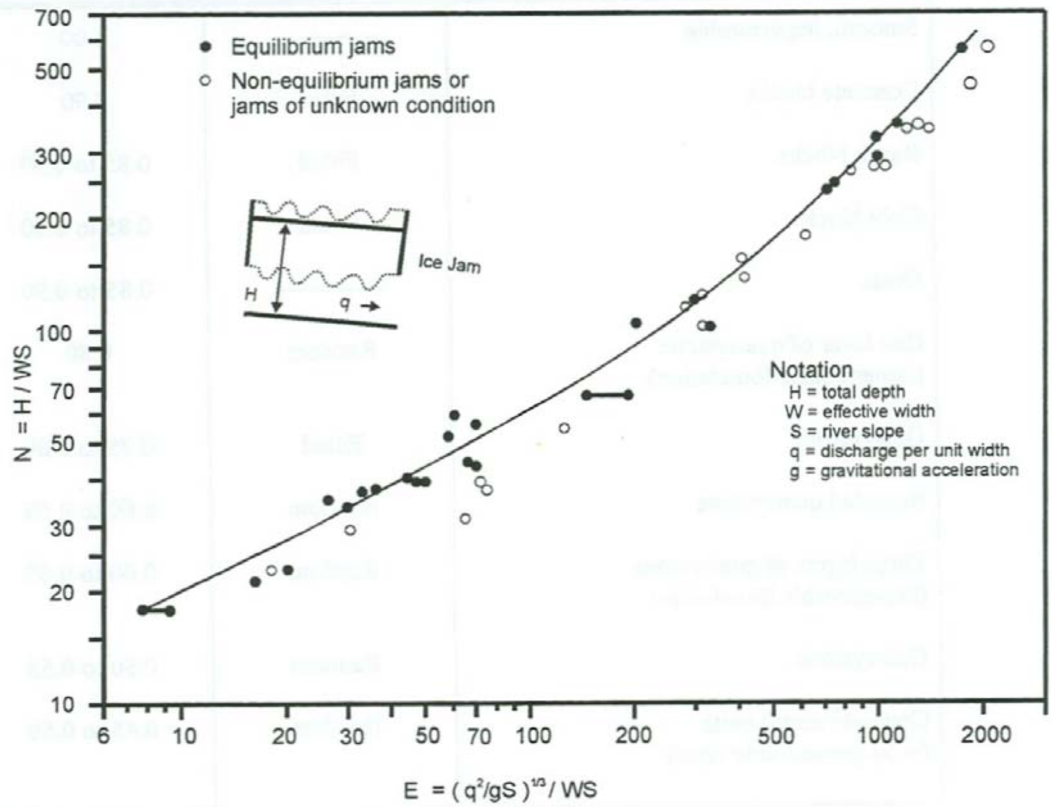
Source: Bender (1995)

Design Chart 5.19: Hydraulic Relationship " *K* "



Source: Bender (1995)

Design Chart 5.20: Fully Developed Ice Jam: Dimensionless Rating Curve



Source: Beltaos (1983)

Design Chart 5.21: Correction Factors for Wave Run-up

Slope Surface Characteristics	Placement	r
Smooth, impermeable	------------	1.00
Concrete blocks	Fitted	0.90
Basalt blocks	Fitted	0.85 to 0.90
Gobi blocks	Fitted	0.85 to 0.90
Grass	------------	0.85 to 0.90
One layer of quarrystone (impermeable foundation)	Random	0.80
Quarrystone	Fitted	0.75 to 0.80
Rounded quarrystone	Random	0.60 to 0.65
Three layers of quarrystone (impermeable foundation)	Random	0.60 to 0.65
Quarrystone	Random	0.50 to 0.55
Concrete armor units (~ 50 percent void ratio)	Random	0.45 to 0.50

Source: U.S. Army Corps of Engineers (1984)

Design Chart 5.22: Suggested K_p for Armour for Wave Protection

No-damage Criteria and Minor Overtopping							
			Structure Trunk		Structure Head		
Armour Units	n^3	Placement	K_D [2]		K_D		Slope
			Breaking Wave	Nonbreaking Wave	Breaking Wave	Nonbreaking Wave	Cot θ
Quarrystone Smooth rounded Smooth rounded Rough angular	2 >3 1	Random Random [4] Random	*1.2* *1.6* [4]	2.4 *3.2* *2.9*	*1.1* *1.4* [4]	1.9 *2.3* *2.3*	1.5 to 3.0 [5] [5]
Rough angular	2	Random	2.0	4.0	*1.9* *1.6* 1.3	3.2 2.8 2.3	1.5 2.0 3.0
Rough angular Rough angular [7] Parallelepiped	>3 2 2	Random [6] Special [1] Special	*2.2* 5.8 7.0 - 20.0	*4.5* 7.0 *8.5 - 24.0*	*2.1* *5.3* ----	*4.2* 6.4	[5] [5]
Tetrapod and Quadripod	2	Random	7.0	8.0	*5.0* *4.5* *3.5*	6.0 5.5 4.0	1.5 2.0 3.0
Tribar	2	Random	*9.0*	10.0	*8.3* *7.8* 6.0	9.0 8.5 6.5	1.5 2.0 3.0
Dolos	2	Random	15.8 [8]	31.8 [8]	*8.0* 7.0	*16.0* *14.0*	2.0 [9] 3.0
Modified Cube Hexapod Toskane Tribar Quarrystone (K_{RR}) Graded Angular	2 2 2 1 -	Random Random Random Uniform Random	*6.5* *8.0* *11.0* 12.0 2.2	7.5 9.5 22.0 15.0 2.5	---- *5.0* ---- *7.5* ----	*5.0* *7.0* ---- *9.5* ----	[5] [5] [5] [5]

1 Caution: Those K_D values shown in *italics* are unsupported by test results and are only provided for preliminary design purposes.

2 Applicable to slopes ranging from 1 on 1.5 to 1 on 5.

3 n is the number of units comprising the thickness of the armour layer.

4 The use of single layer of quarrystone armour units is not recommended for structures subject to breaking waves, and only under special conditions for structures subject to nonbreaking waves. When it is used, the stone should be carefully placed.

5 Until more information is available on the variation of K_D value with slope, the use of K_D should be limited to slopes ranging from 1on 1.5 to 1 on 3. Some armour units tested on a structure head indicate a K_D - slope dependence.

6 Special placement with long axis of stone placed perpendicular to structure face.

7 Parallelepiped-shaped stone: long slab-like stone with the dimension about 3 times the shortest dimension (Markle and Davidson, 1979).

8 Refers to no-damage criteria (<5 percent displacement, rocking, etc.); if no rocking (<2 percent) is desired, reduce K_D 50 percent (Zwamborn and Van Niekerk, 1982).

9 Stability of Dolosse on slopes steeper than 1 on 2 should be substantiated by site-specific model tests.

Source: U.S. Army Corps of Engineers (1984)

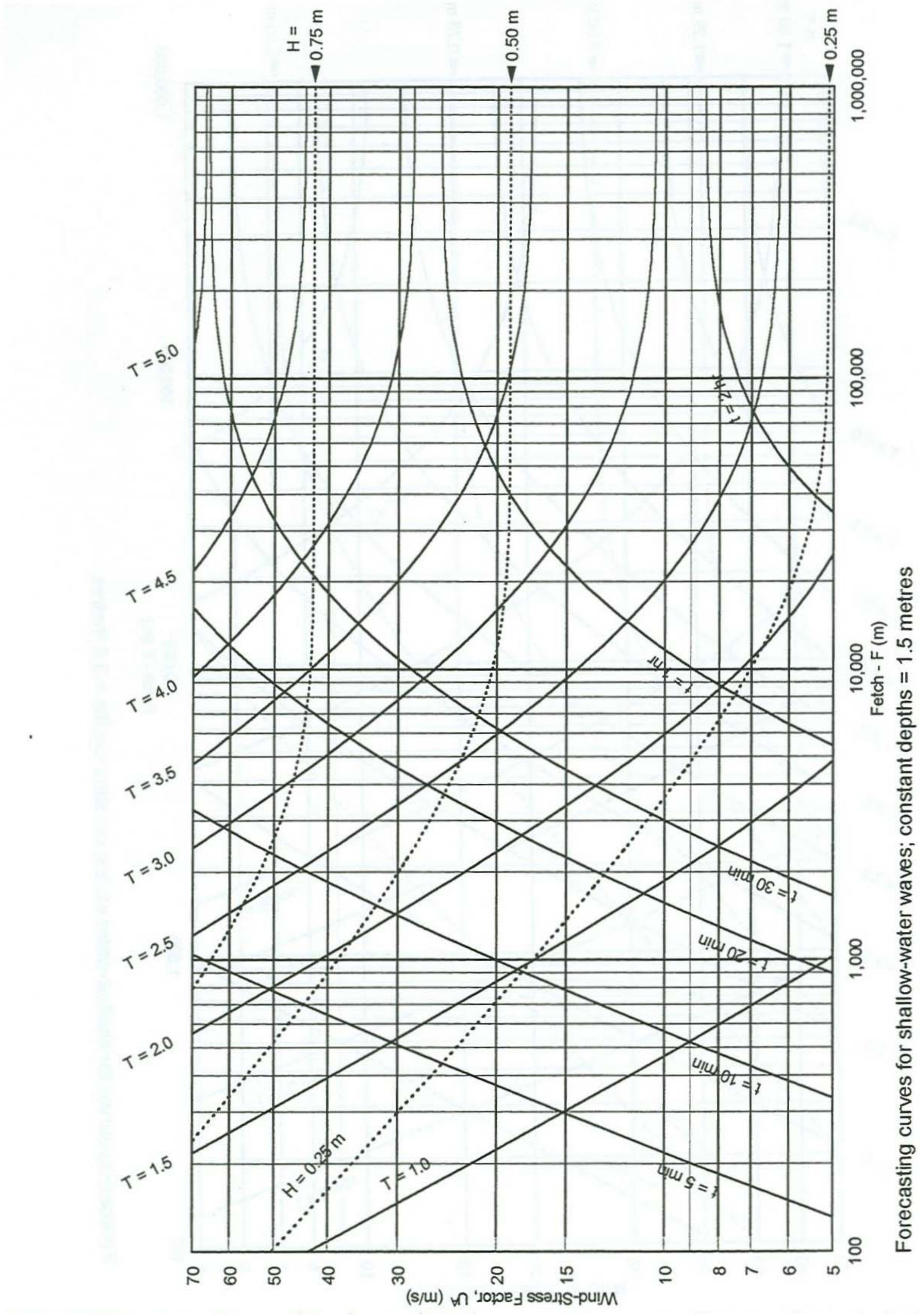
Design Chart 5.23: Layer Coefficient and Porosity for Armour for Wave Protection

Armour Unit	n	Placement	Layer Coefficient k_Δ	Porosity (P) %
QuarryStone (smooth) [1]	2	Random	1.02	38
QuarryStone (rough) [2]	2	Random	1.00	37
QuarryStone (rough) [2]	> 3	Random	1.00	40
QuarryStone (parallelepiped) [6]	2	Special	-----	27
Cube (modified) [1]	2	Random	1.10	47
Tetrapod [1]	2	Random	1.04	50
Quadripod [1]	2	Random	0.95	49
Hexipod [1]	2	Random	1.15	47
Tribar [1]	2	Random	1.02	54
Dolos [4]	2	Random	0.94	56
Toskane [5]	2	Random	1.03	52
Tribar [1]	1	Uniform	1.13	47
Quarrystone [7]	Graded	Random	-----	37

1 Hudson (1974).
2 Carver (1983).
3 Hudson (1961a).
4 Carver and Davidson (1977).
5 Carver (1978).
6 Layer thickness is twice the average long dimension of the parallelepiped stones. Porosity is estimated from tests on one layer of uniformly placed modified cubes.
7 The minimum layer thickness should be twice the cubic dimension of the W_{50} riprap. Check to determine that the graded layer thickness is $\geq$ 1.25 the cubic dimension of the W_{max} riprap.

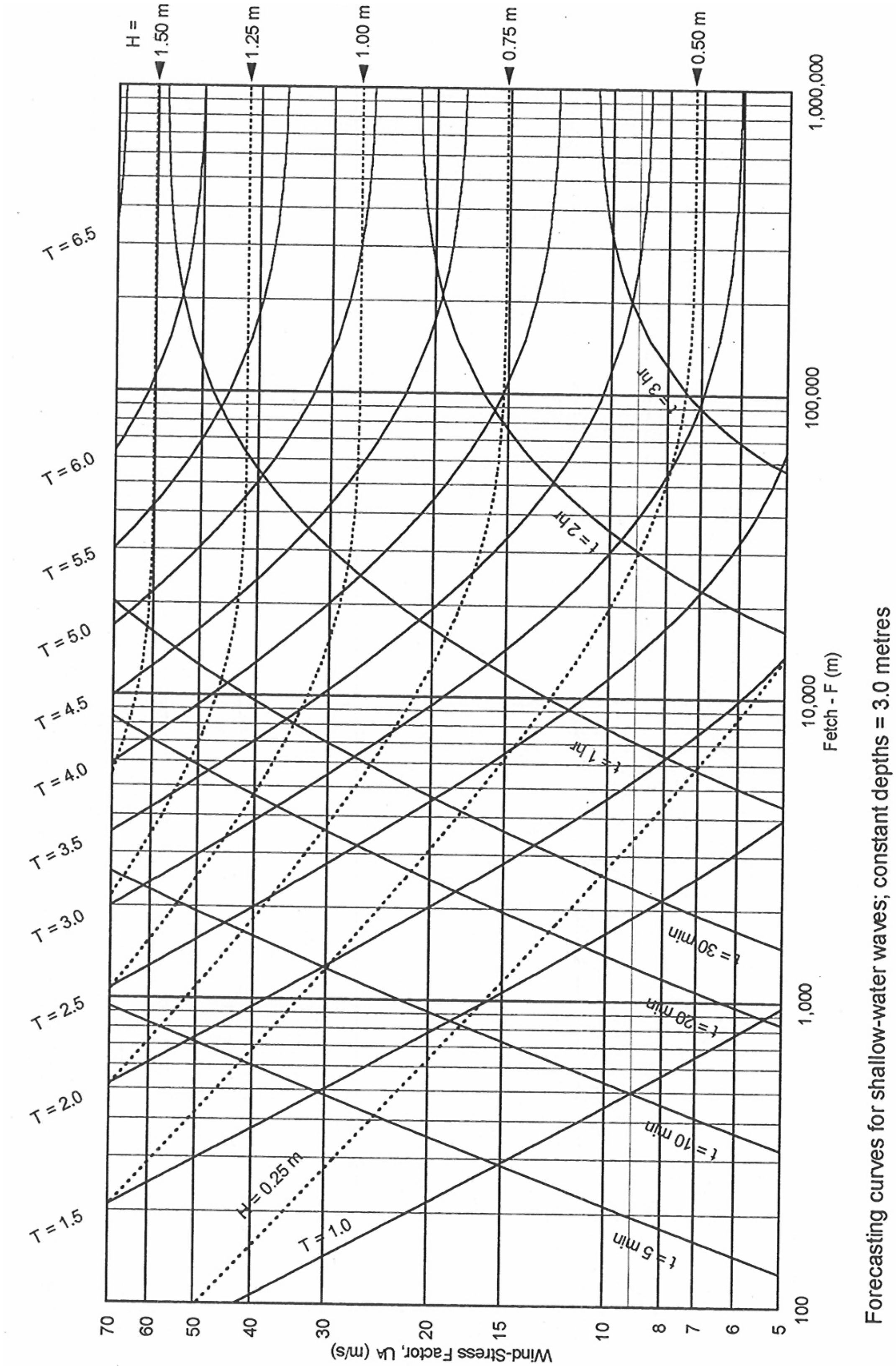
Source: U.S. Army Corps of Engineers (1984)

Design Chart 5.24: Forecasting Curves for Waves



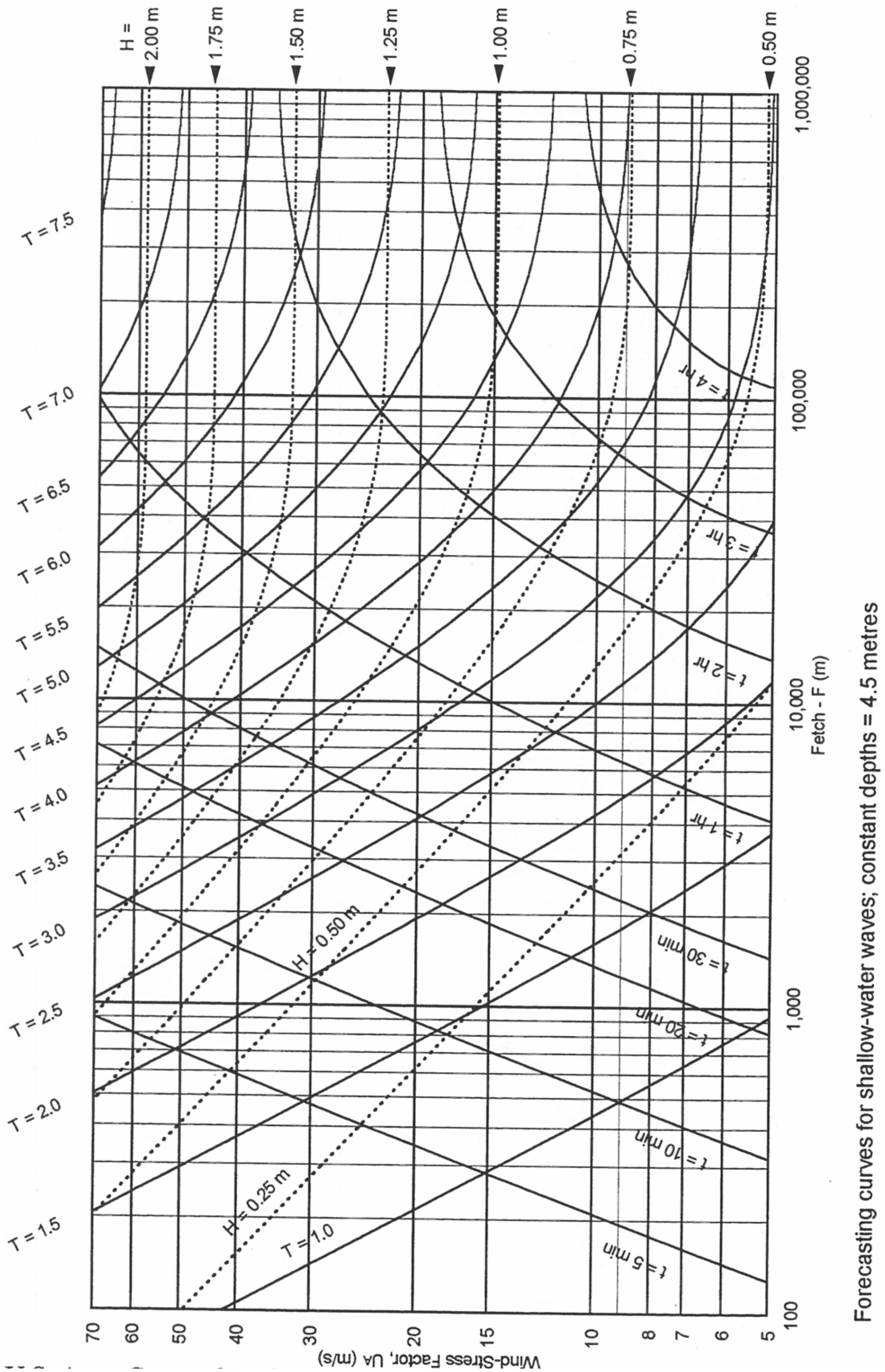
Source: U.S. Army Corps of Engineers (1984)

Design Chart 5.25: Forecasting Curves for Waves



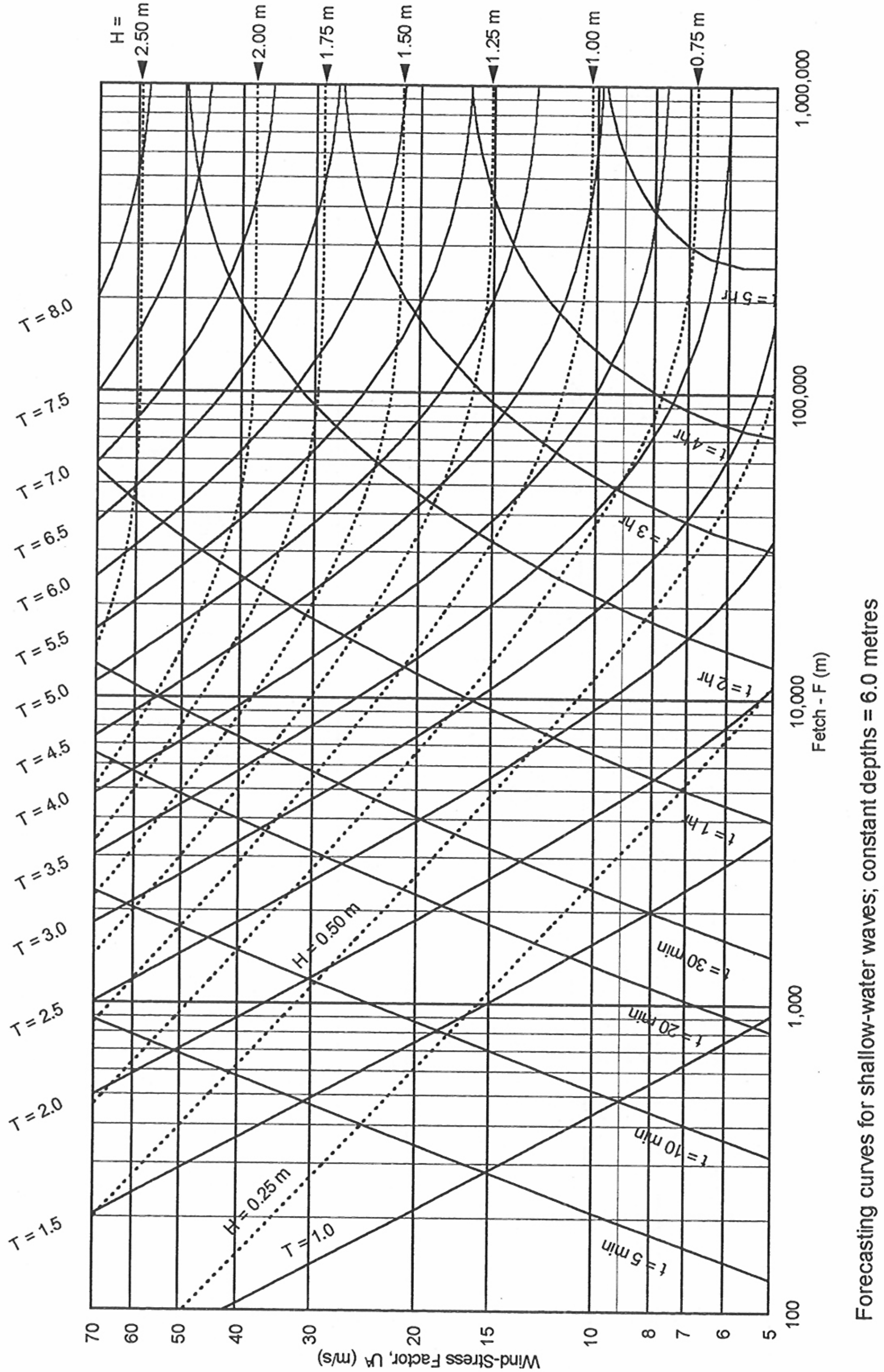
Source: U.S. Army Corps of Engineers (1984)

Design Chart 5.26: Forecasting Curves for Waves



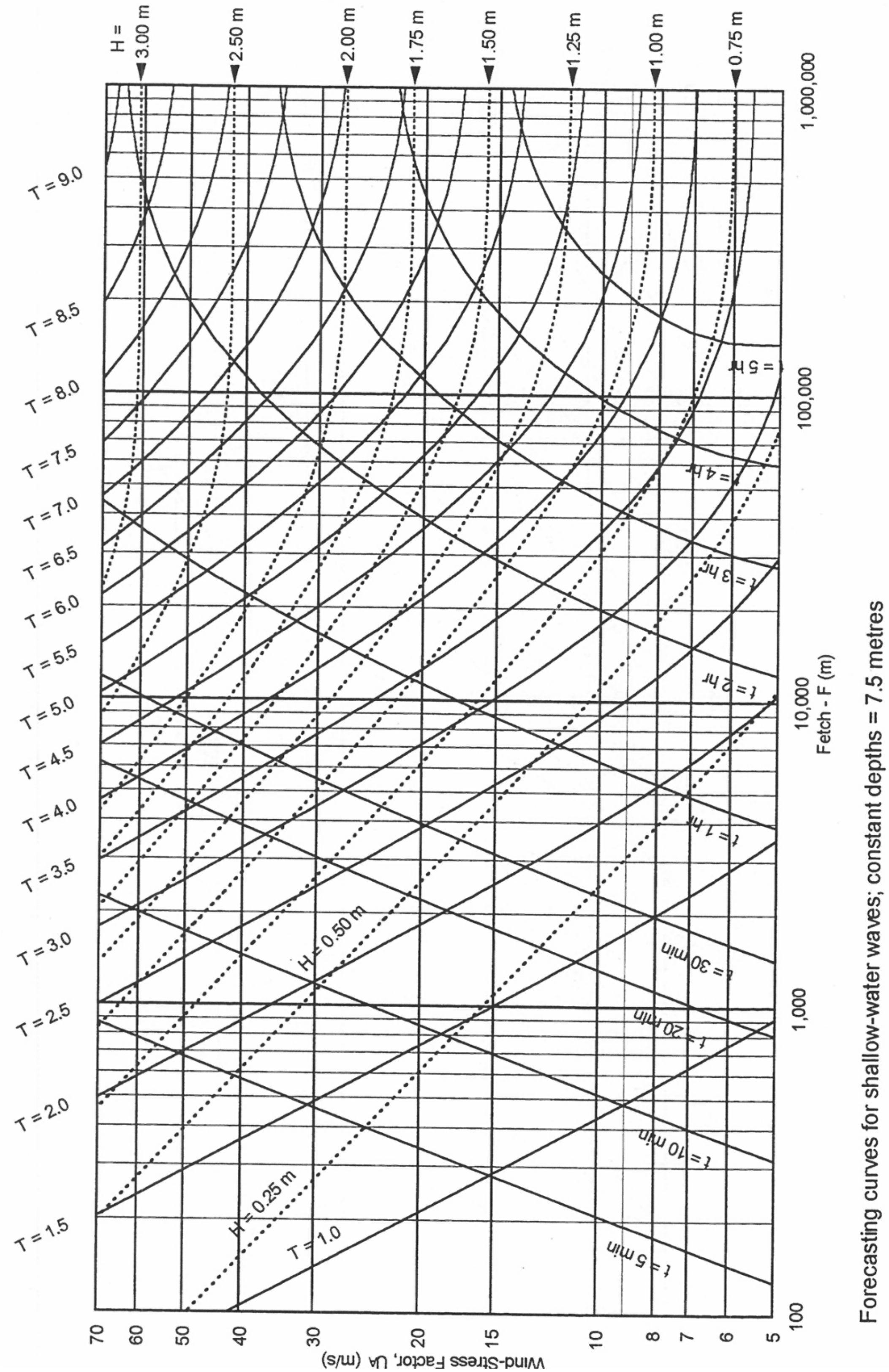
Source: U.S Army Corps of Engineers (1984)

Design Chart 5.27: Forecasting Curves for Waves



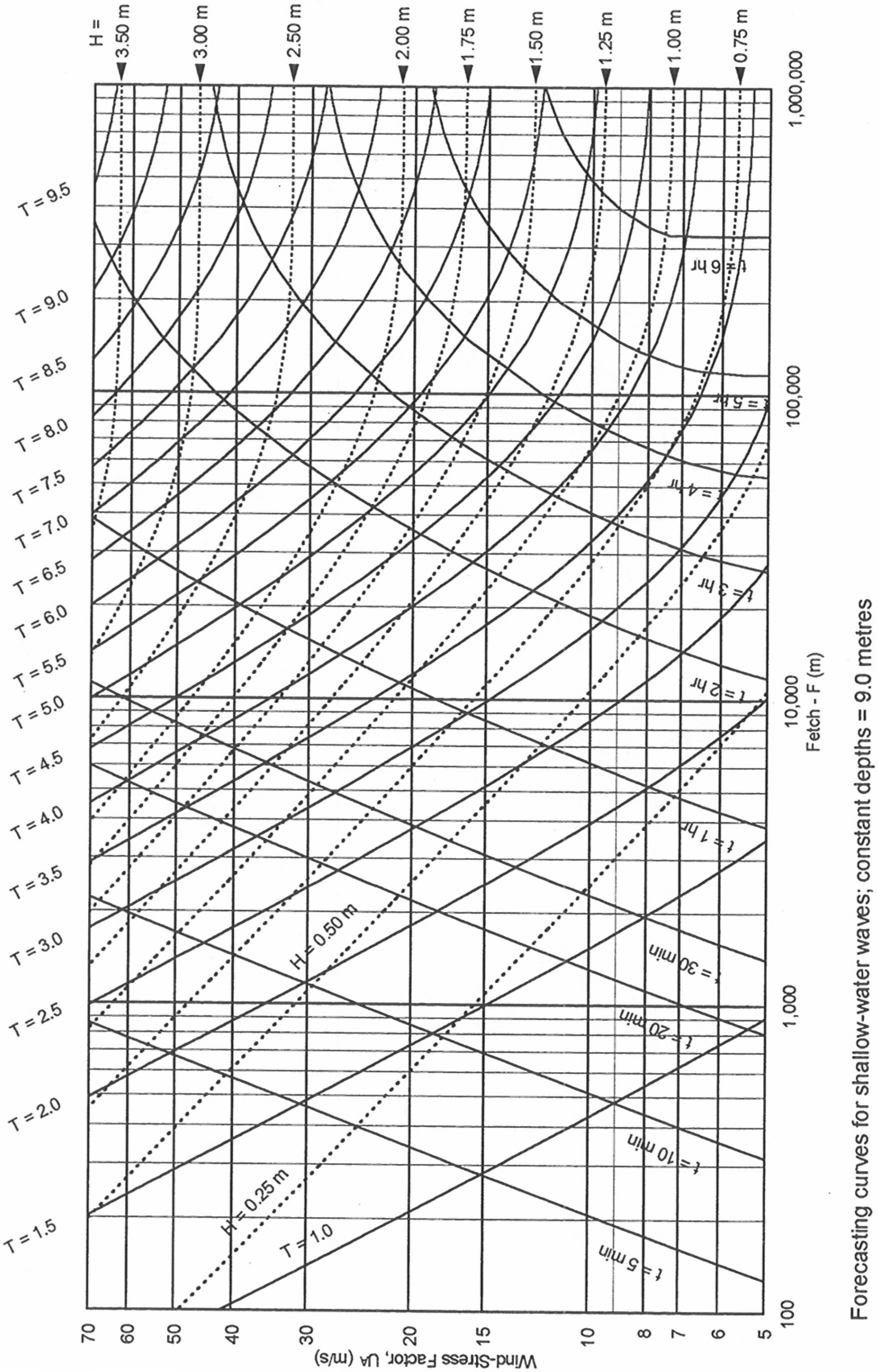
Source: U.S. Army Corps of Engineers (1984)

Design Chart 5.28: Forecasting Curves for Waves



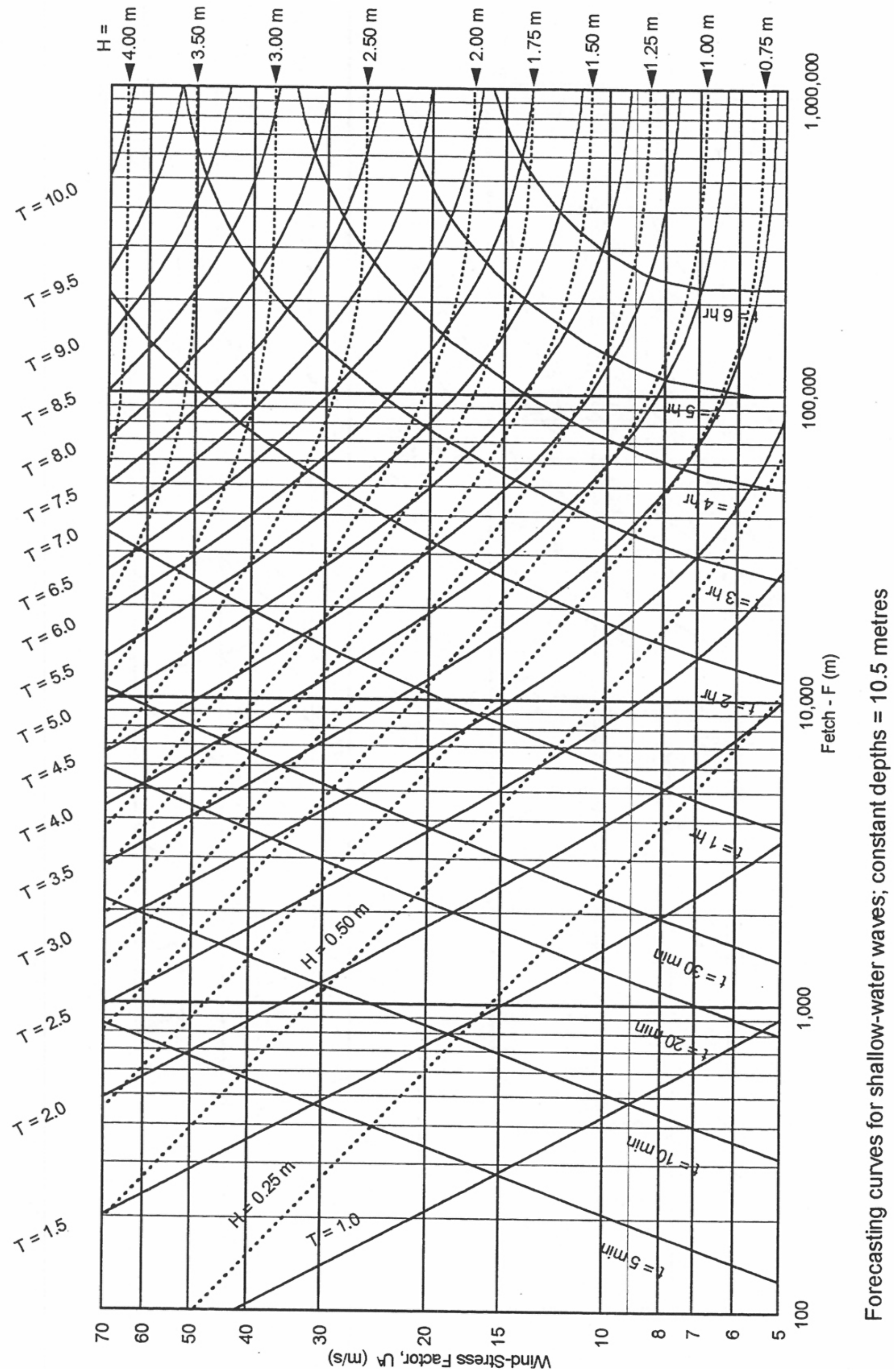
Source: U.S. Army Corps of Engineers (1984)

Design Chart 5.29: Forecasting Curves for Waves



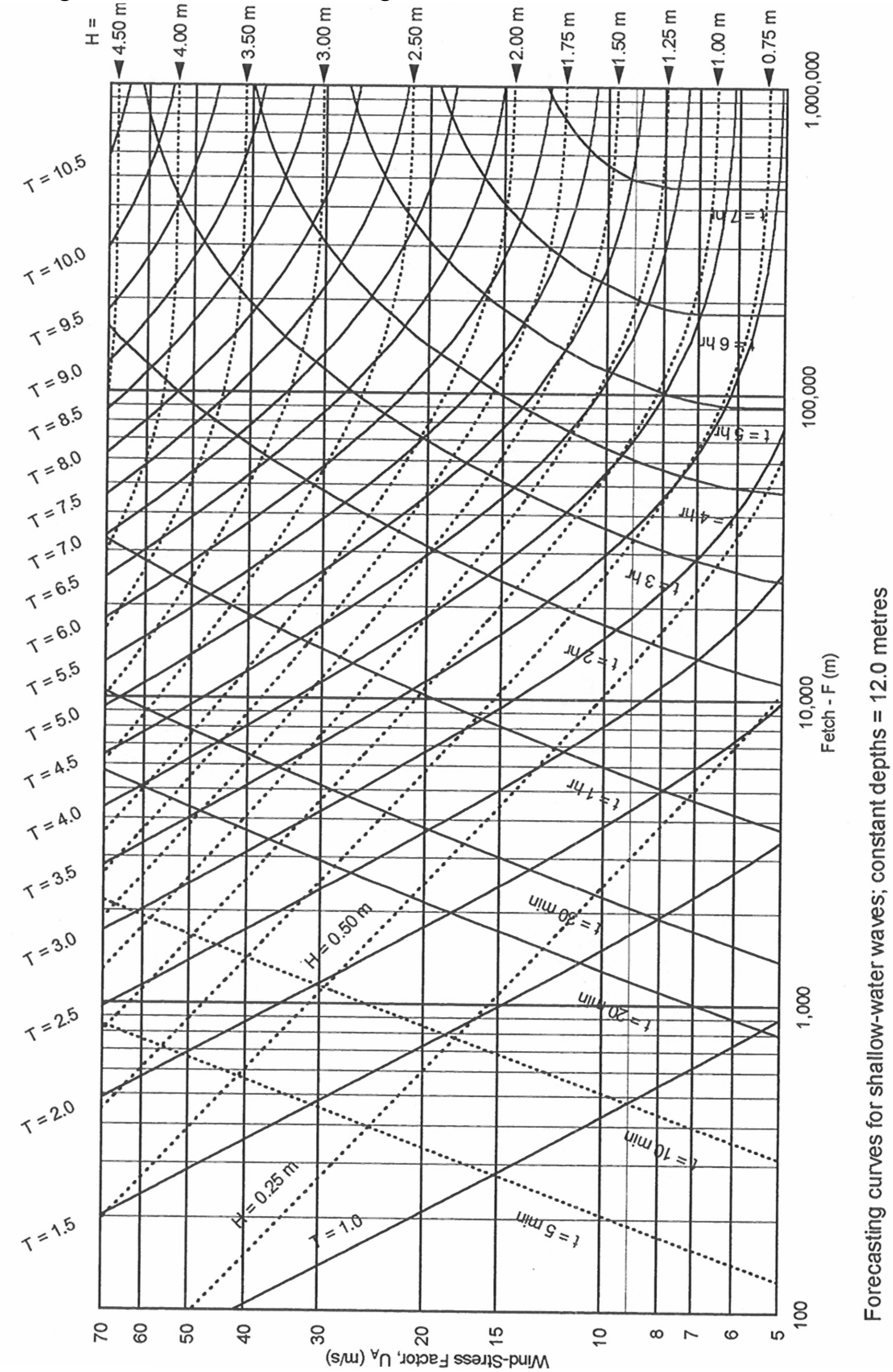
Source: U.S. Army Corps of Engineers (1984)

Design Chart 5.30: Forecasting Curves for Waves



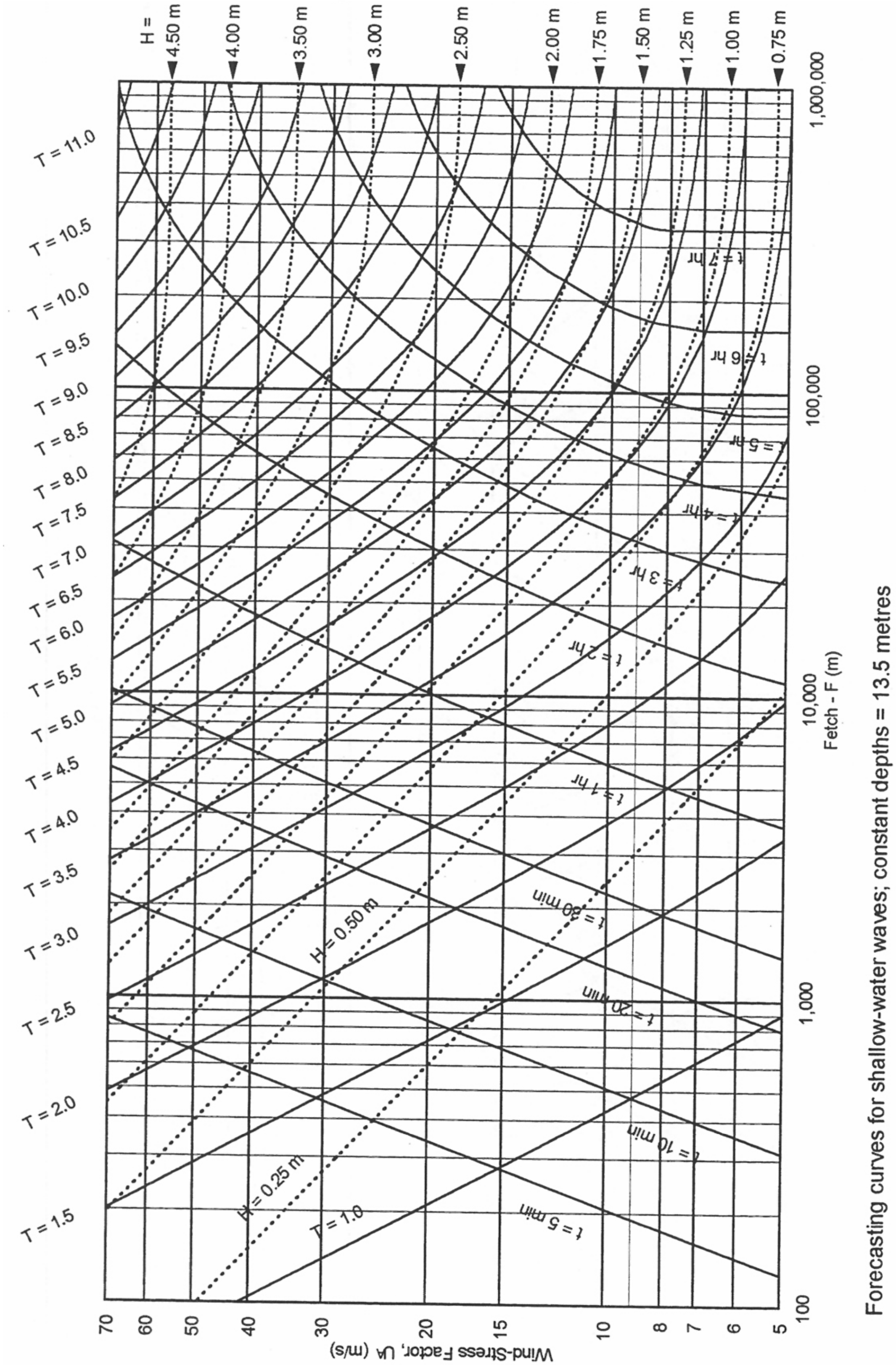
Source: U.S. Army Corps of Engineers (1984)

Design Chart 5.31: Forecasting Curves for Waves



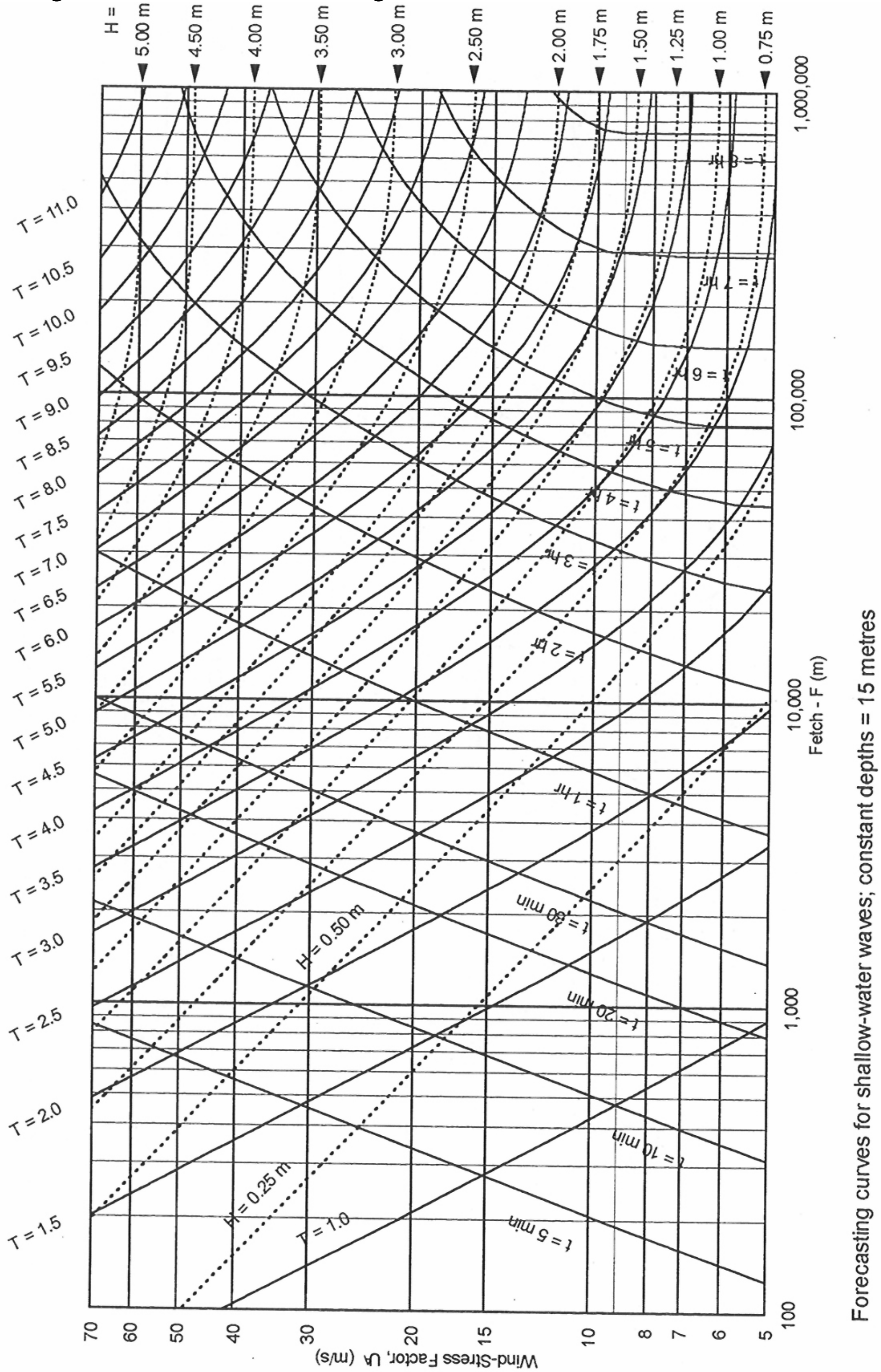
Source: U.S. Army Corps of Engineers (1984)

Design Chart 5.32: Forecasting Curve for Waves



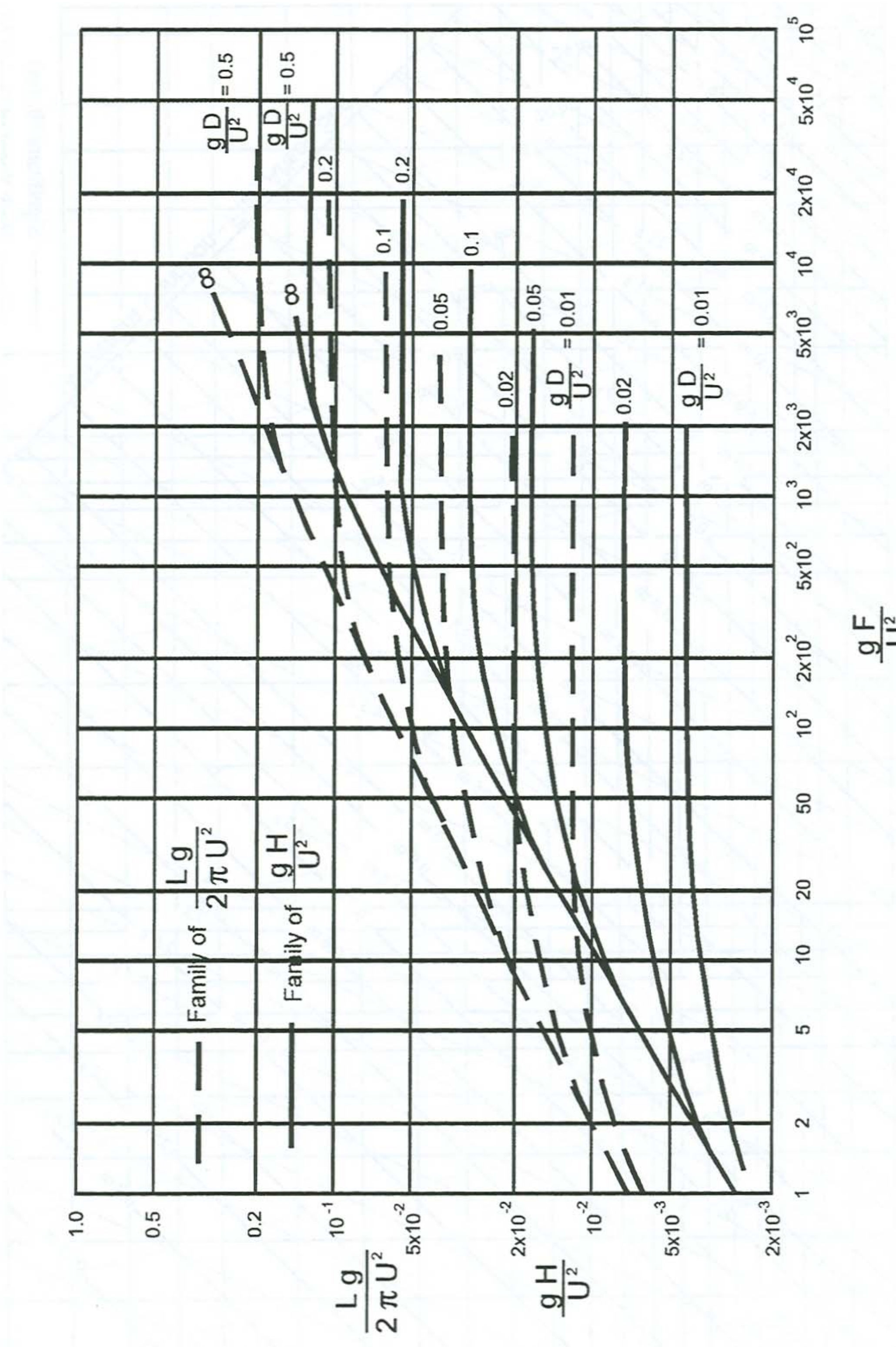
Source: U.S. Army Corps of Engineers (1984)

Design Chart 5.33: Forecasting Curves for Waves



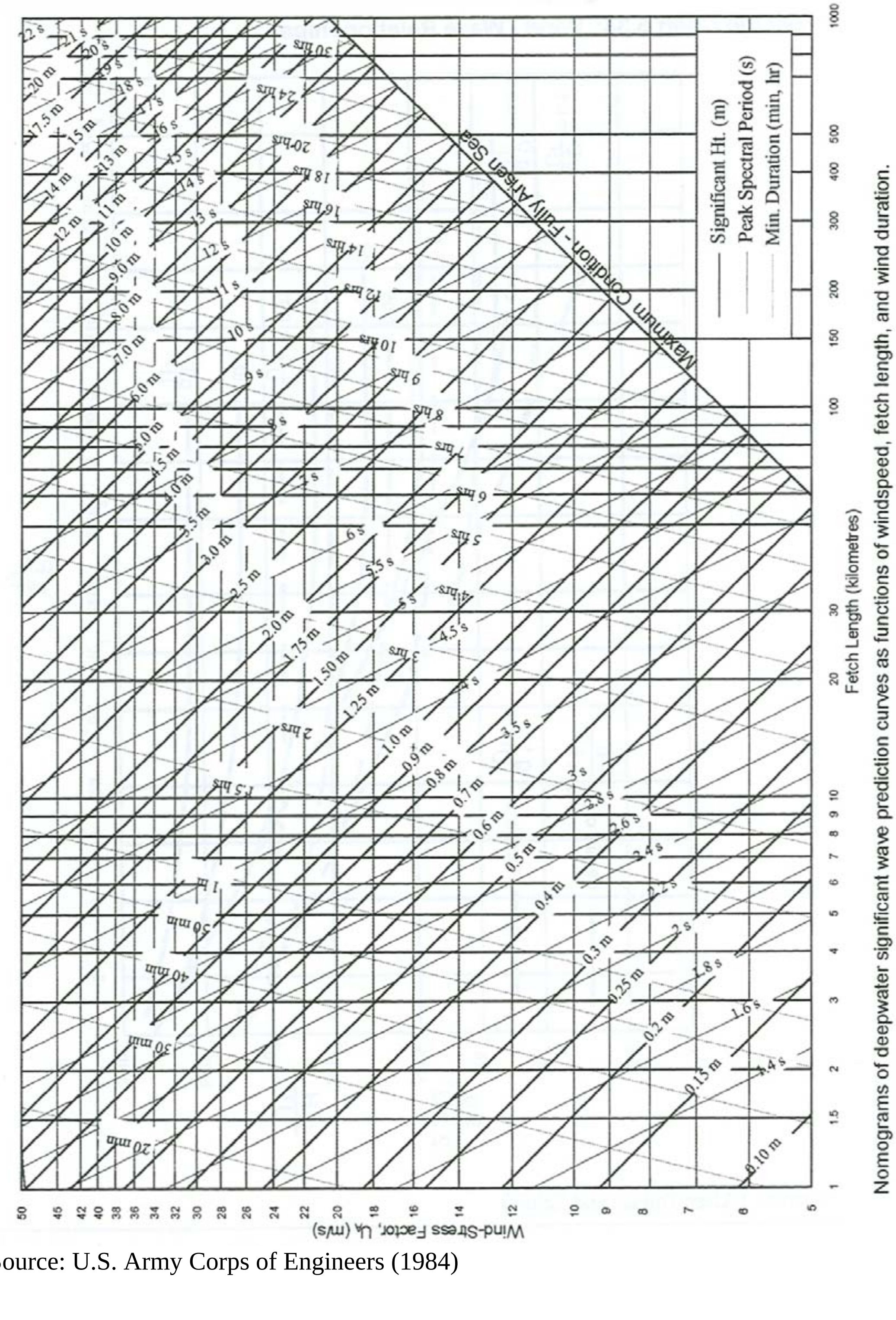
Source: U.S. Army Corps of Engineers (1984)

Design Chart 5.34: Wind - Wave Relationships



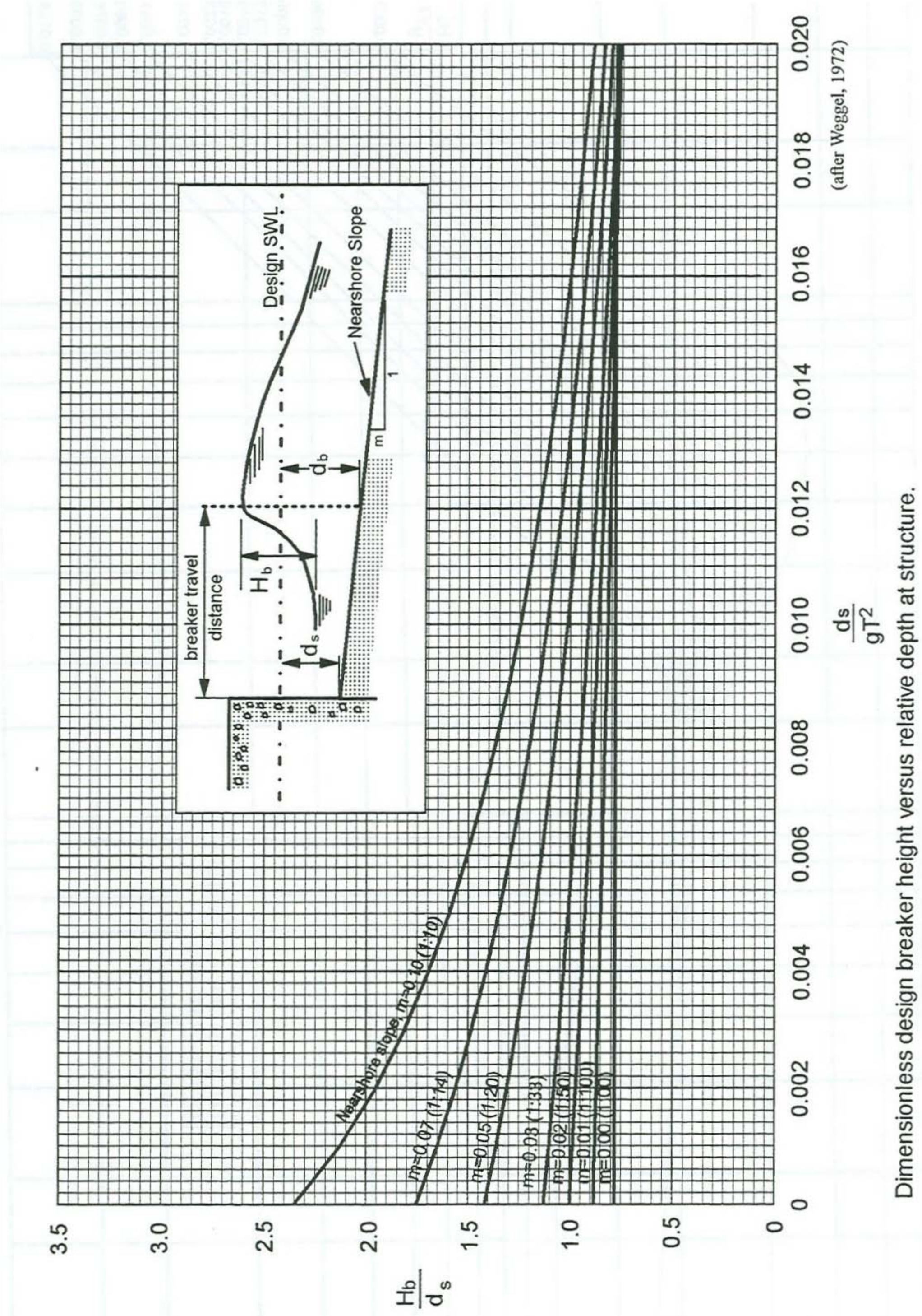
Source: (After Thijsse and Schijf)

Design Chart 5.35: Significant Waves Prediction Curves



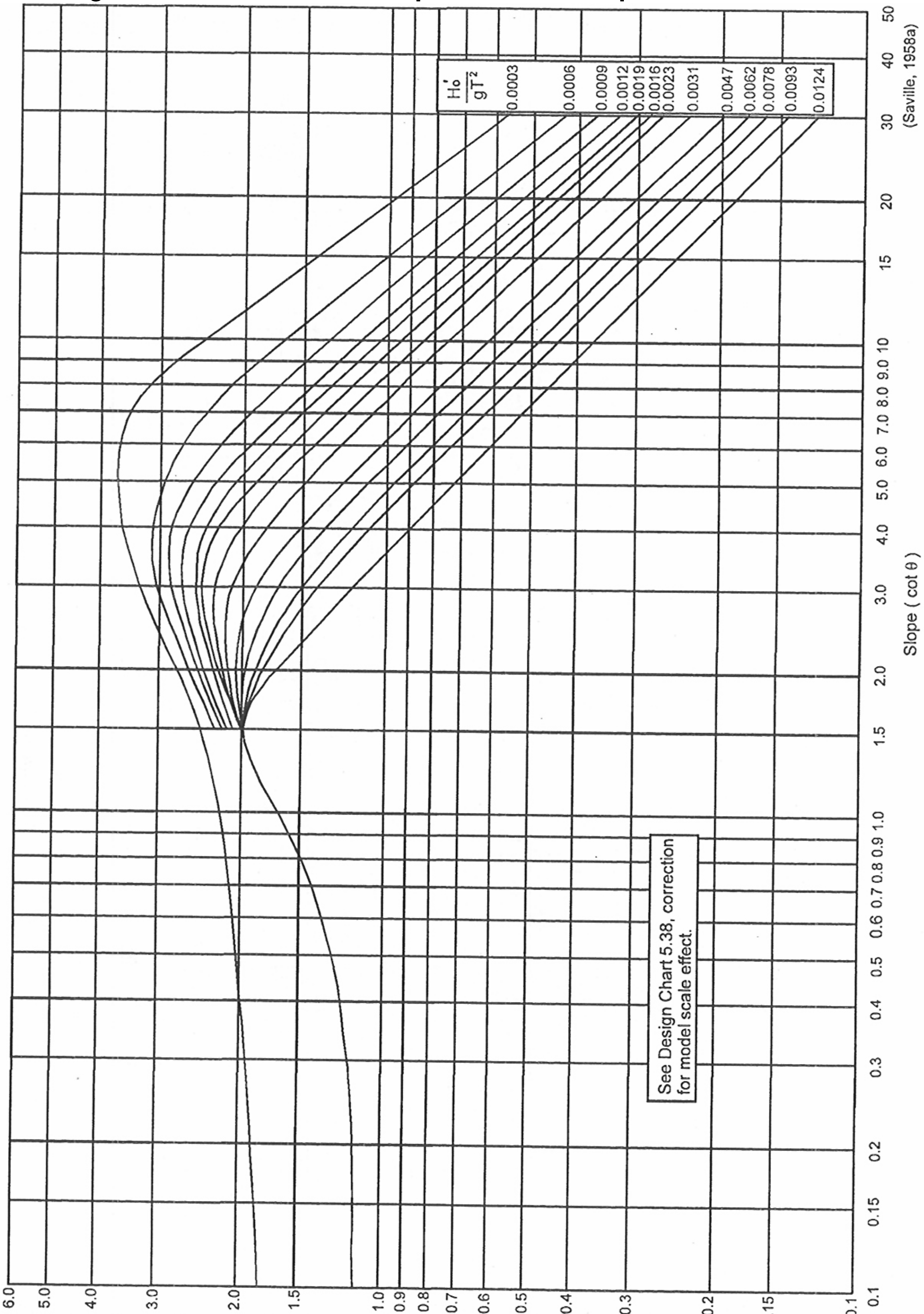
Source: U.S. Army Corps of Engineers (1984)

Design Chart 5.36: Dimensionless Breaker Height v.s. Depth Relationship



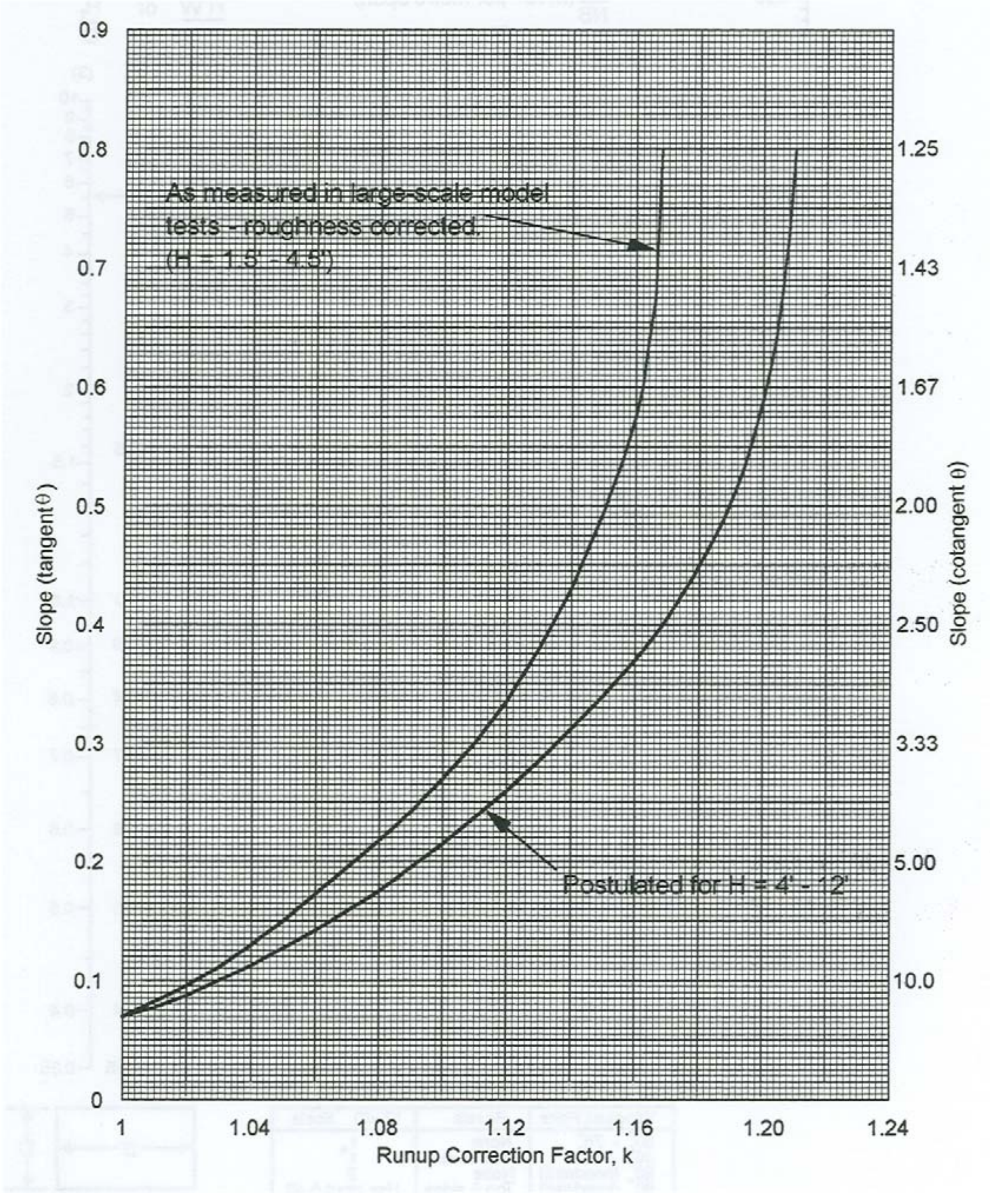
Source: U.S. Army Corps of Engineers (1984)

Design Chart 5.37: Wave Run-up on Smooth Slopes



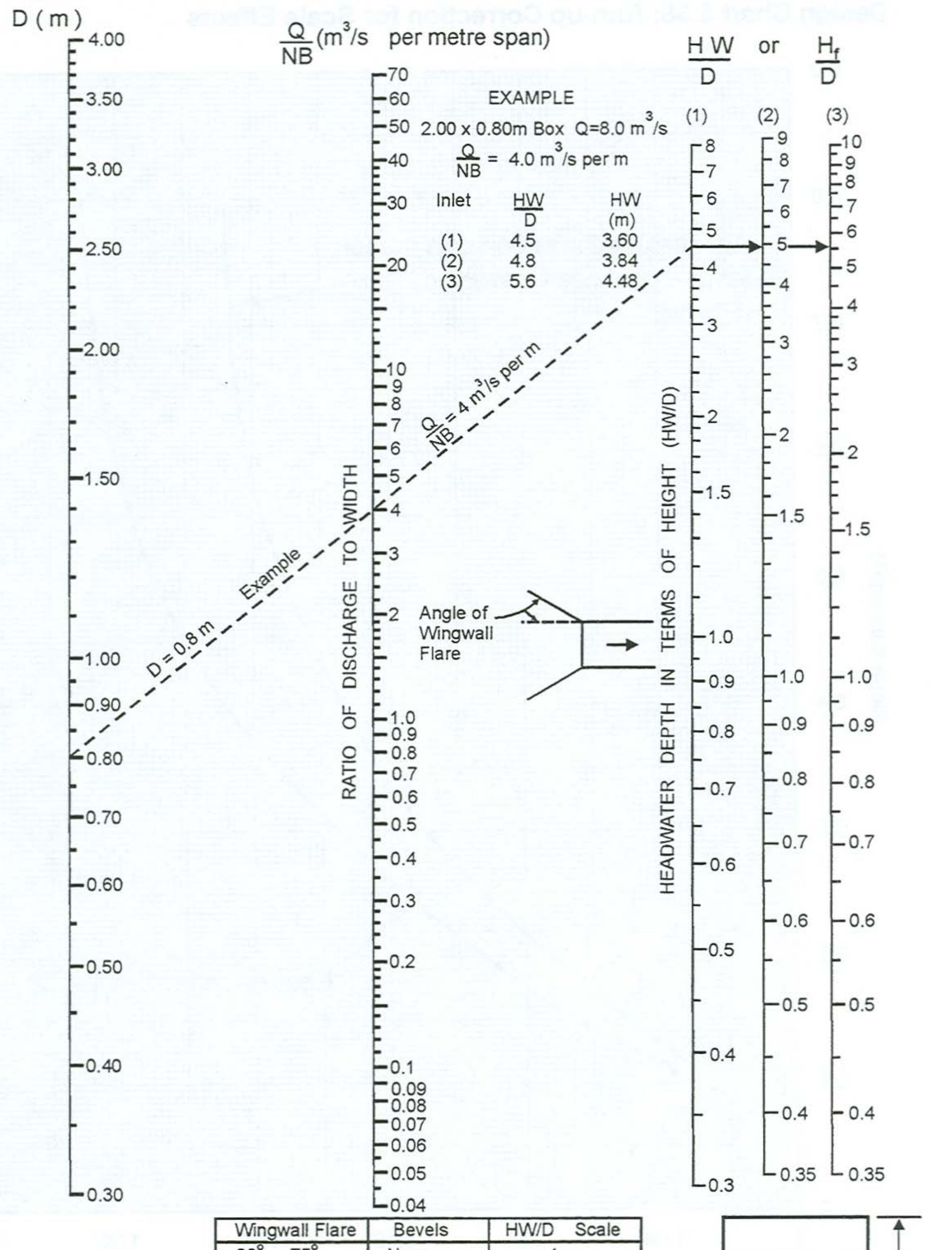
Source: U.S. Army Corps of Engineers (1972)

Design Chart 5.38: Run-up Correction for Scale Effects



Source: U.S. Army Corps of Engineers (1984)

Design Chart 5.39: Inlet Control: Box Culvert

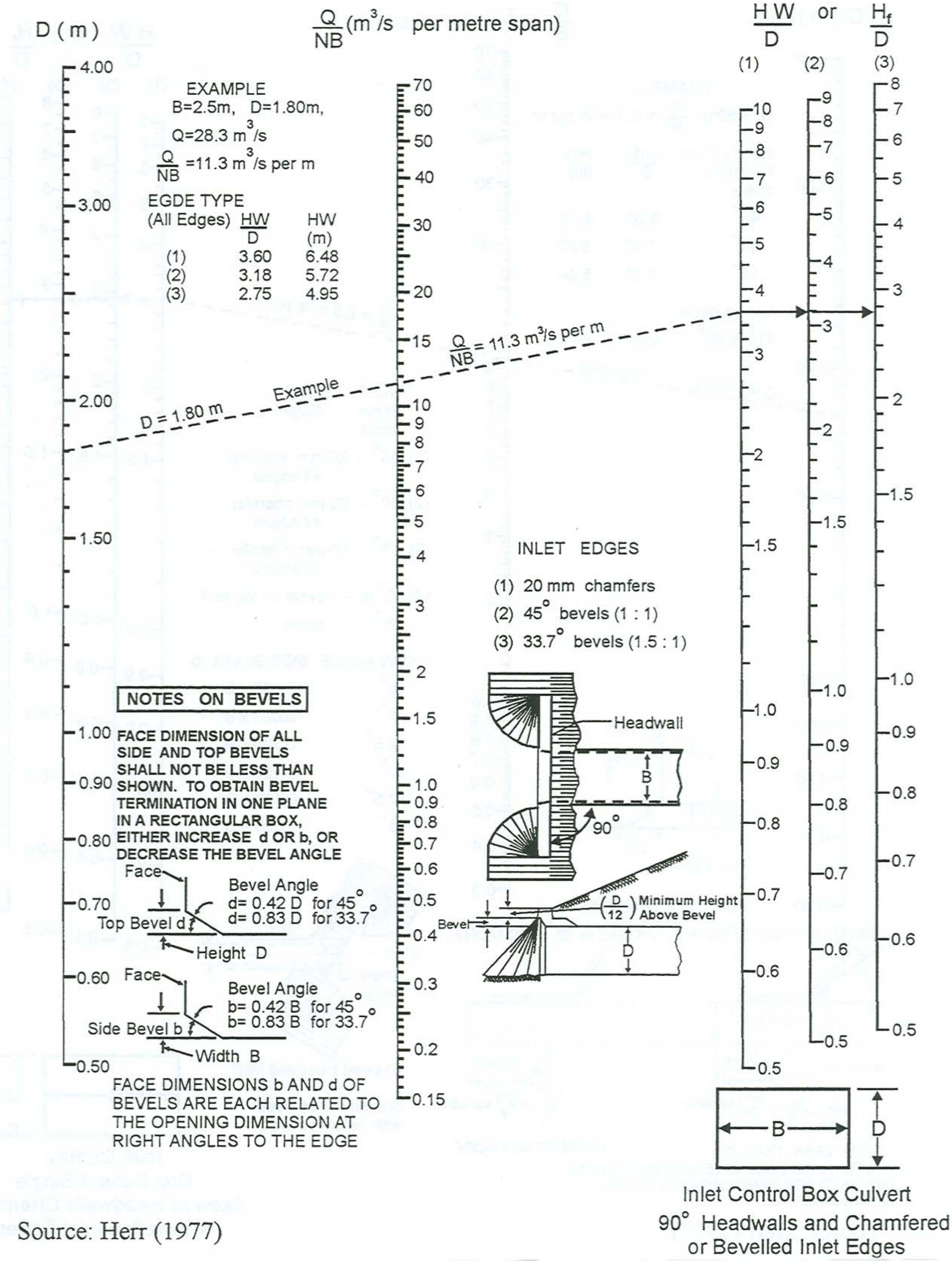


Wingwall Flare	Bevels	HW/D Scale
30° - 75°	None	1
30° - 75°	Top Bevel	1*
90° (headwall)	None	2
90° (headwall)	Top & sides	Use chart 5.40
0° (parallel)	Top Bevel	2
0° (parallel)	None	3

Source: Herr (1977)

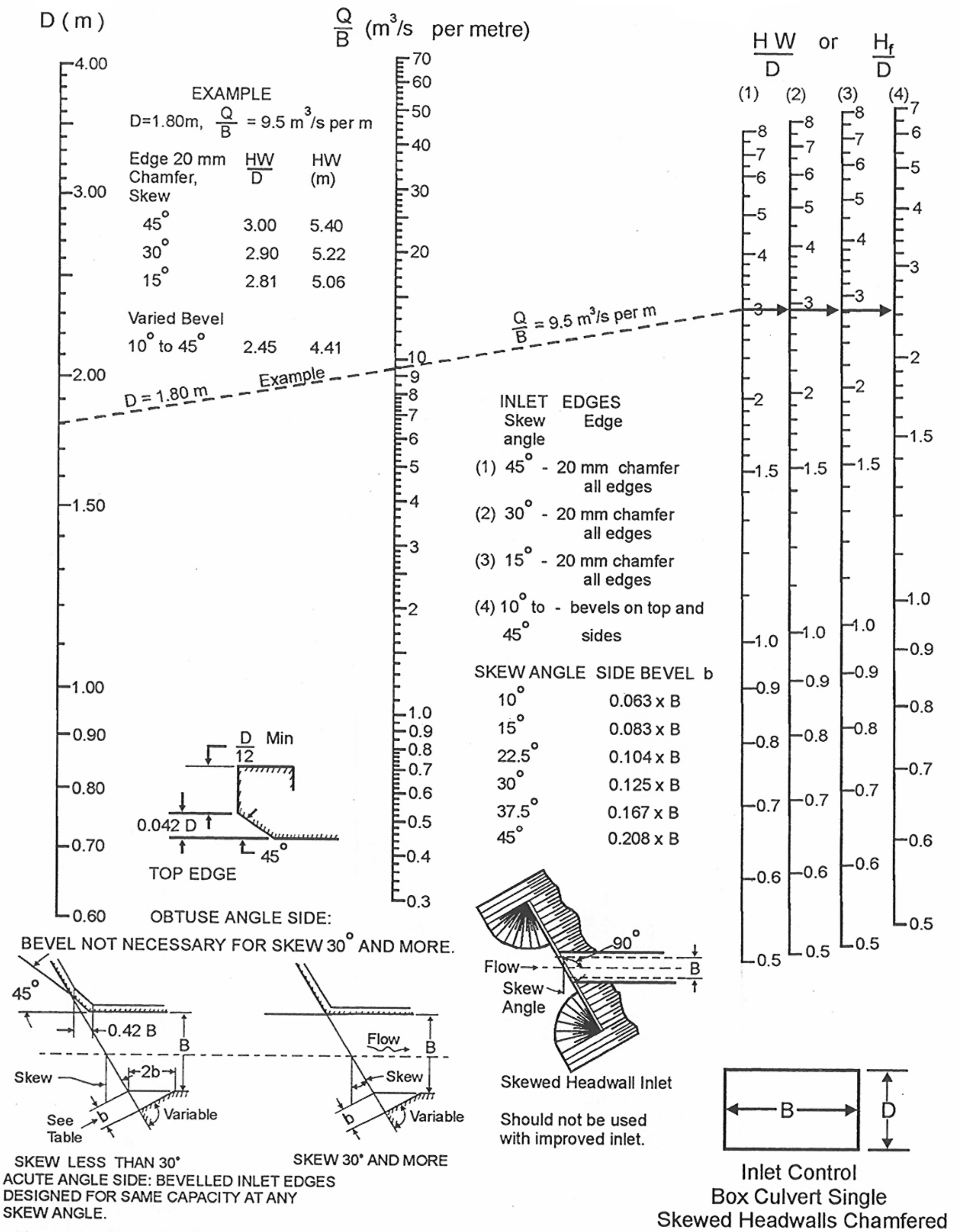
See Design Chart 5.42 also for flares 18° - 45°

Design Chart 5.40: Inlet Control: Box Culverts with Chamfered/Bevelled Edges



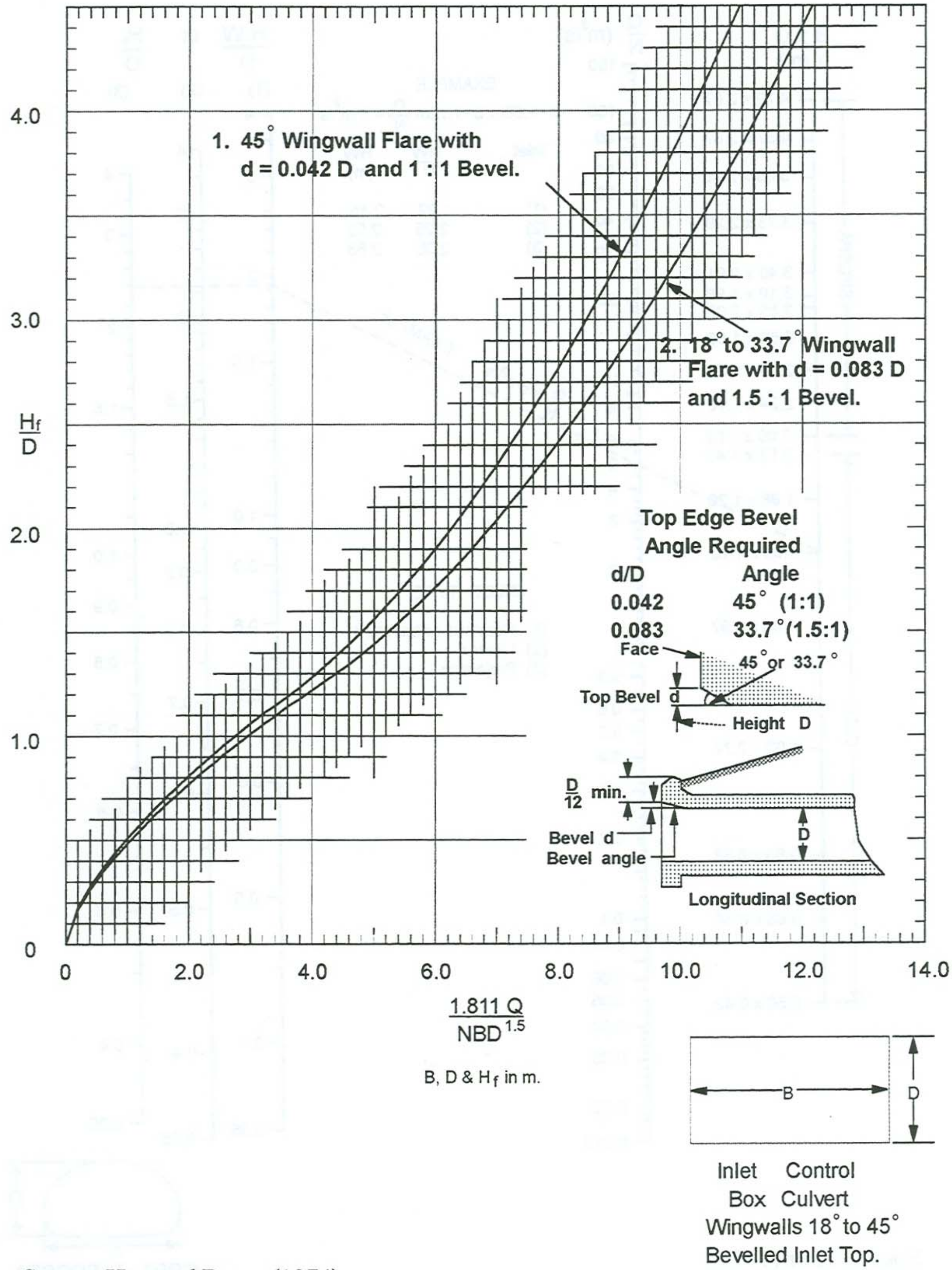
Source: Herr (1977)

Design Chart 5.41: Inlet Control: Box Culverts, Skewed Headwalls



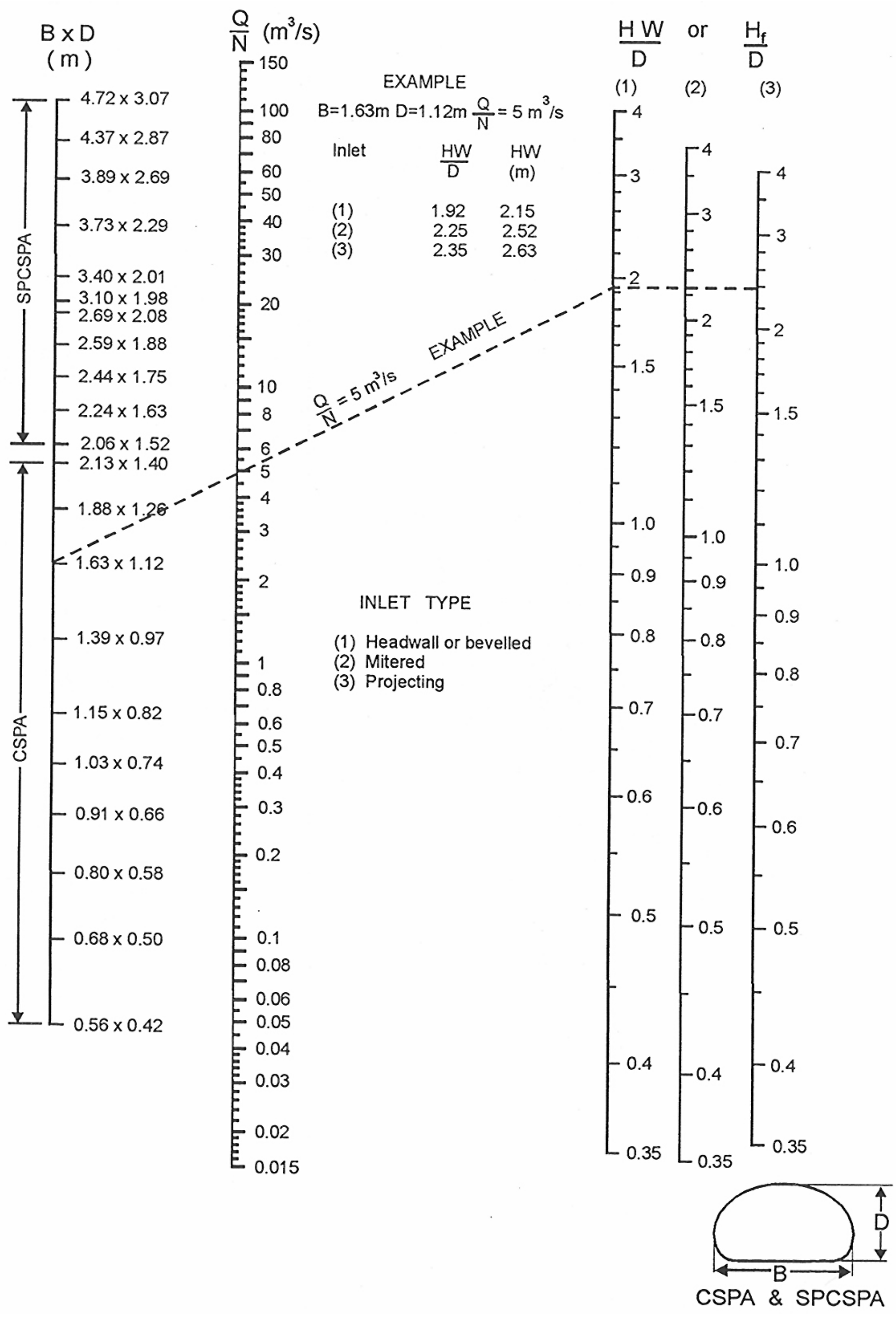
Source: Herr (1977)

Design Chart 5.42: Inlet Control Box Culverts ,Wing Walls



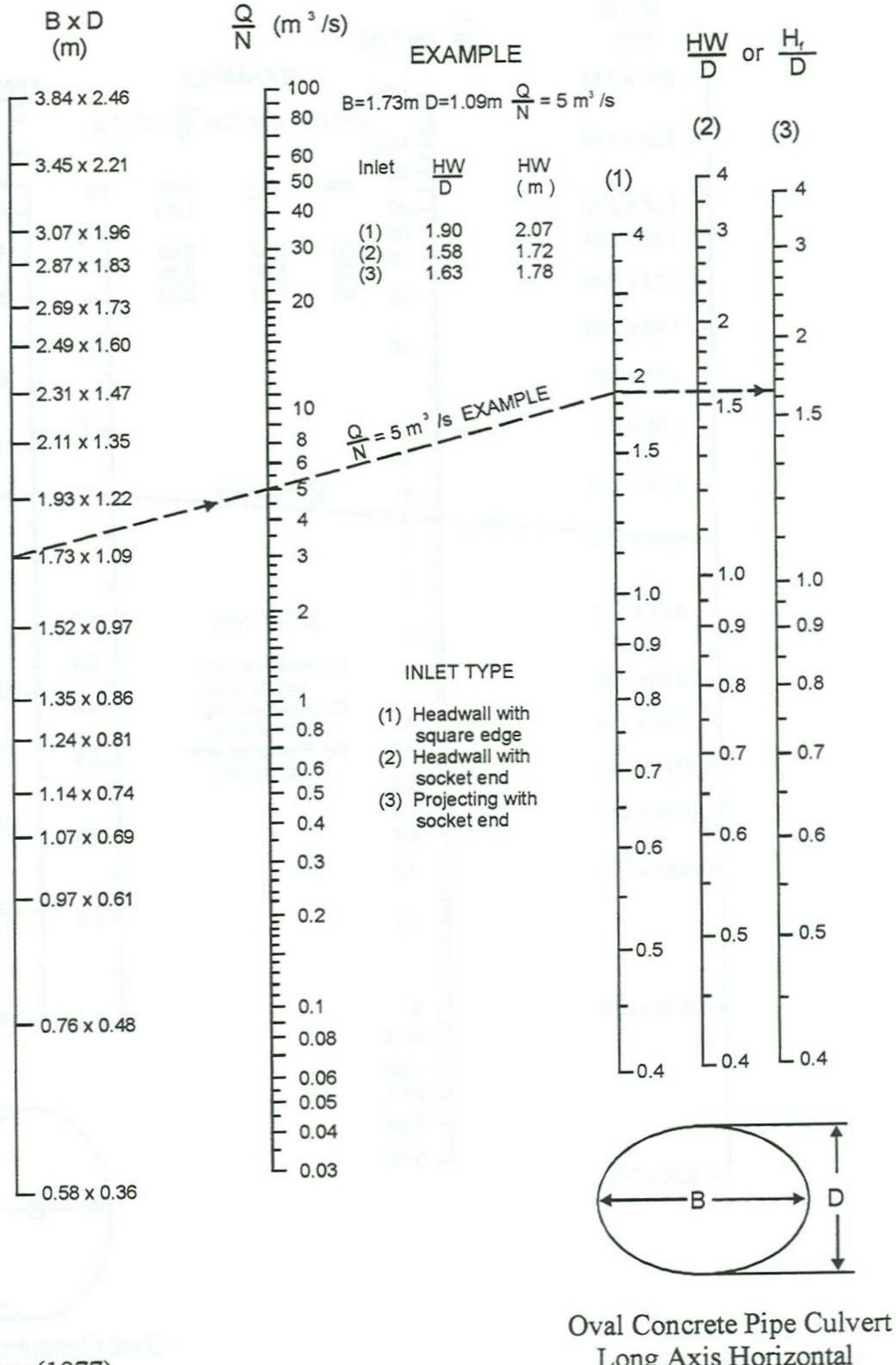
Source: Herr and Bossy (1974)

Design Chart 5.43: Inlet Control: Steel Pipe Arch Culverts



Source: Herr (1977)

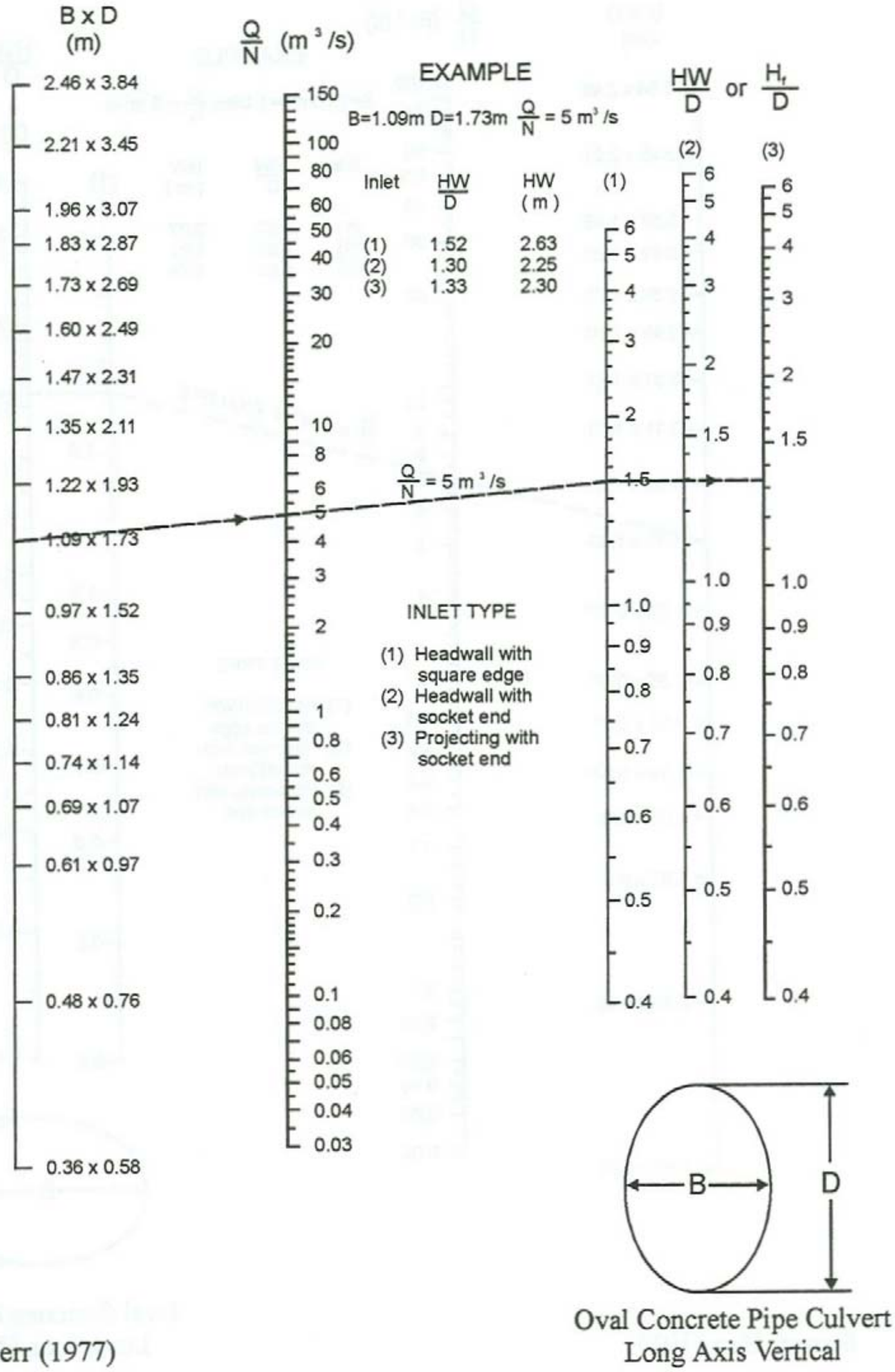
Design Chart 5.44: Inlet Control: Concrete Horizontal Ellipse Culverts



Oval Concrete Pipe Culvert
Long Axis Horizontal

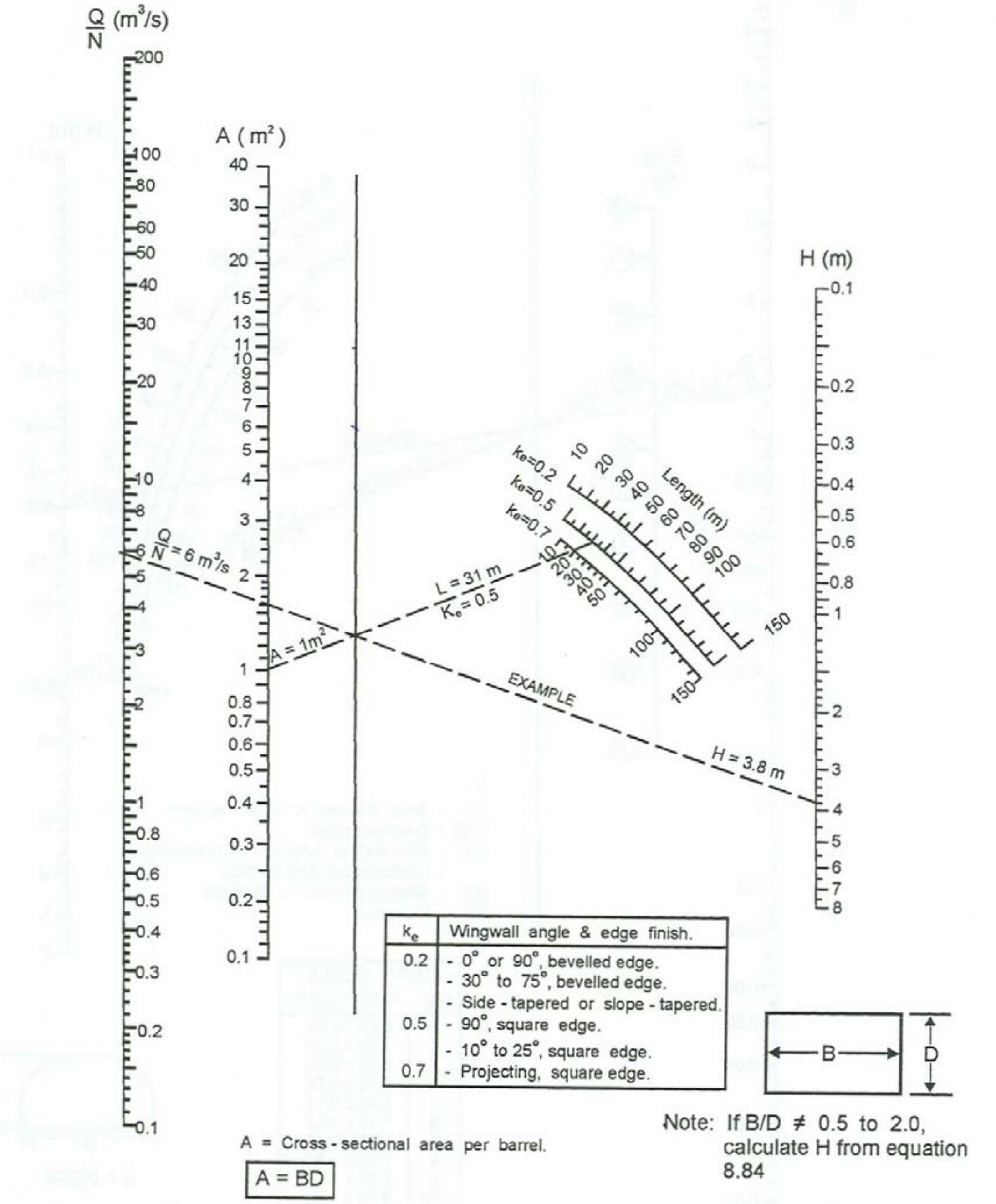
Source: Herr (1977)

Design Chart 5.45: Inlet Control: Concrete Vertical Ellipse Culverts



Source: Herr (1977)

Design Chart 5.46: Outlet Control: Concrete Box Culvert Flowing Full

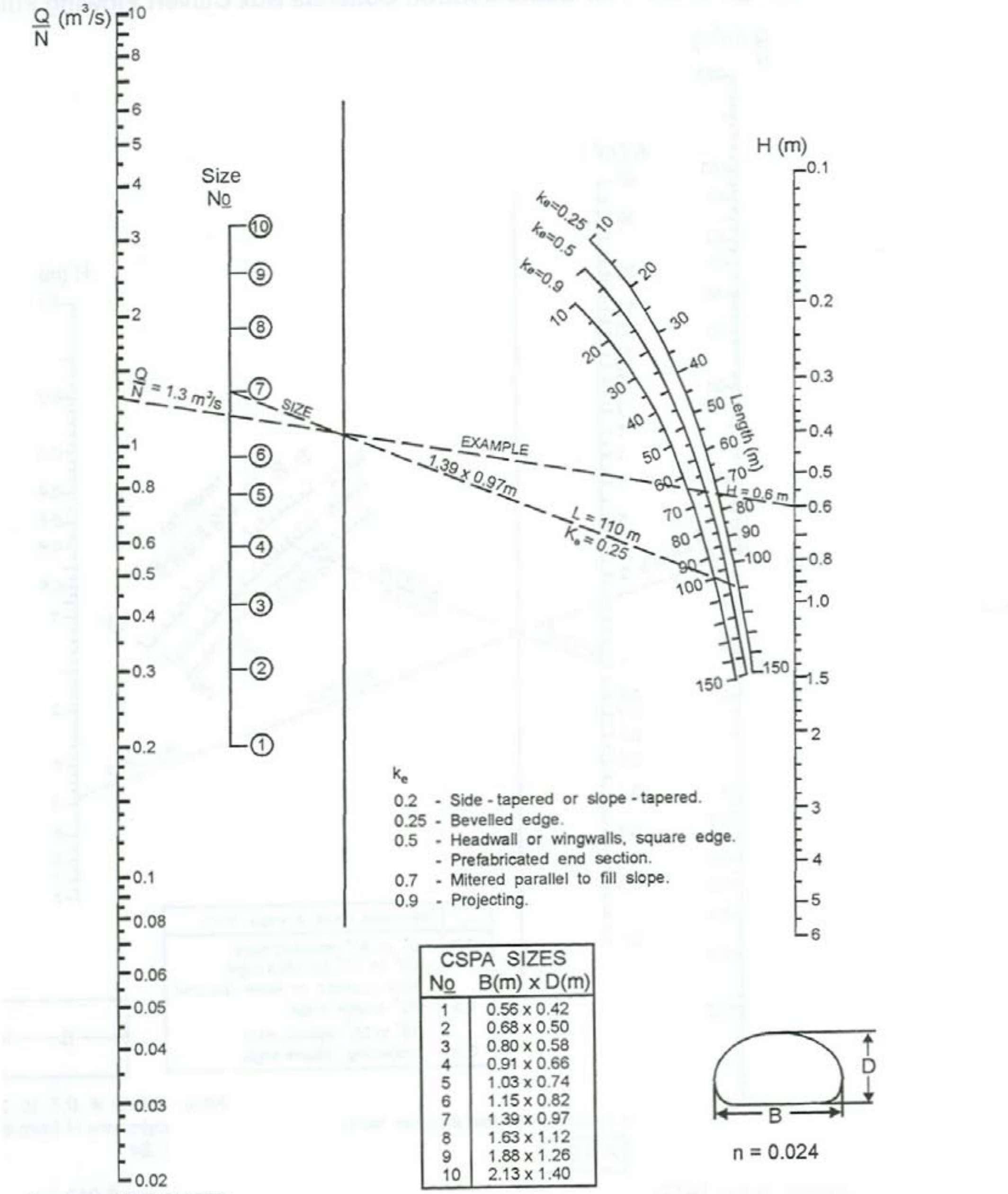


k_e	Wingwall angle & edge finish.
0.2	- 0° or 90°, bevelled edge. - 30° to 75°, bevelled edge. - Side - tapered or slope - tapered.
0.5	- 90°, square edge. - 10° to 25°, square edge.
0.7	- Projecting, square edge.

Source: Herr (1977)

n = 0.012

Design Chart 5.47: Outlet Control: Pipe Arch CSP Culvert - Flowing Full

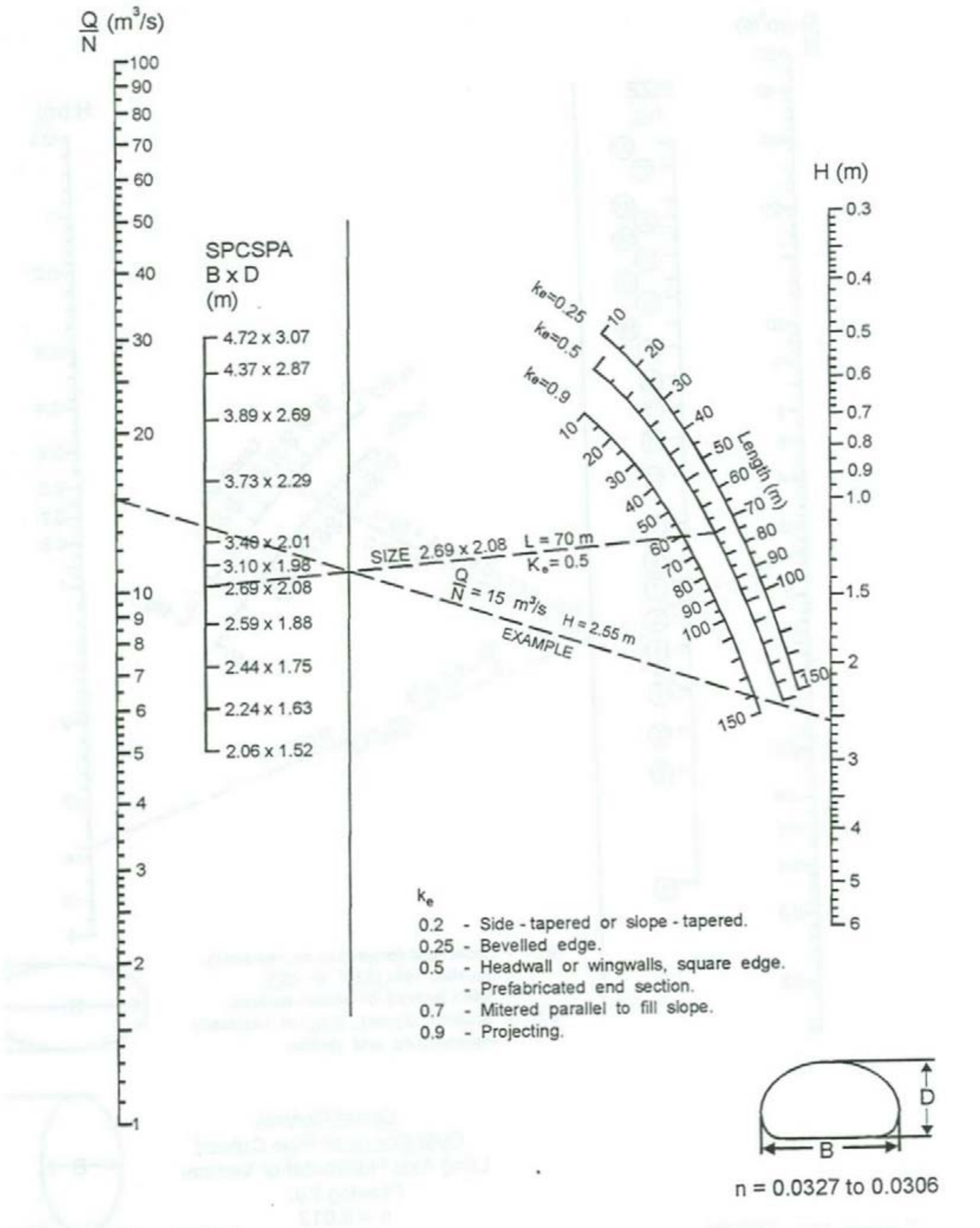


CSPA SIZES

No	B(m) x D(m)
1	0.56 x 0.42
2	0.68 x 0.50
3	0.80 x 0.58
4	0.91 x 0.66
5	1.03 x 0.74
6	1.15 x 0.82
7	1.39 x 0.97
8	1.63 x 1.12
9	1.88 x 1.26
10	2.13 x 1.40

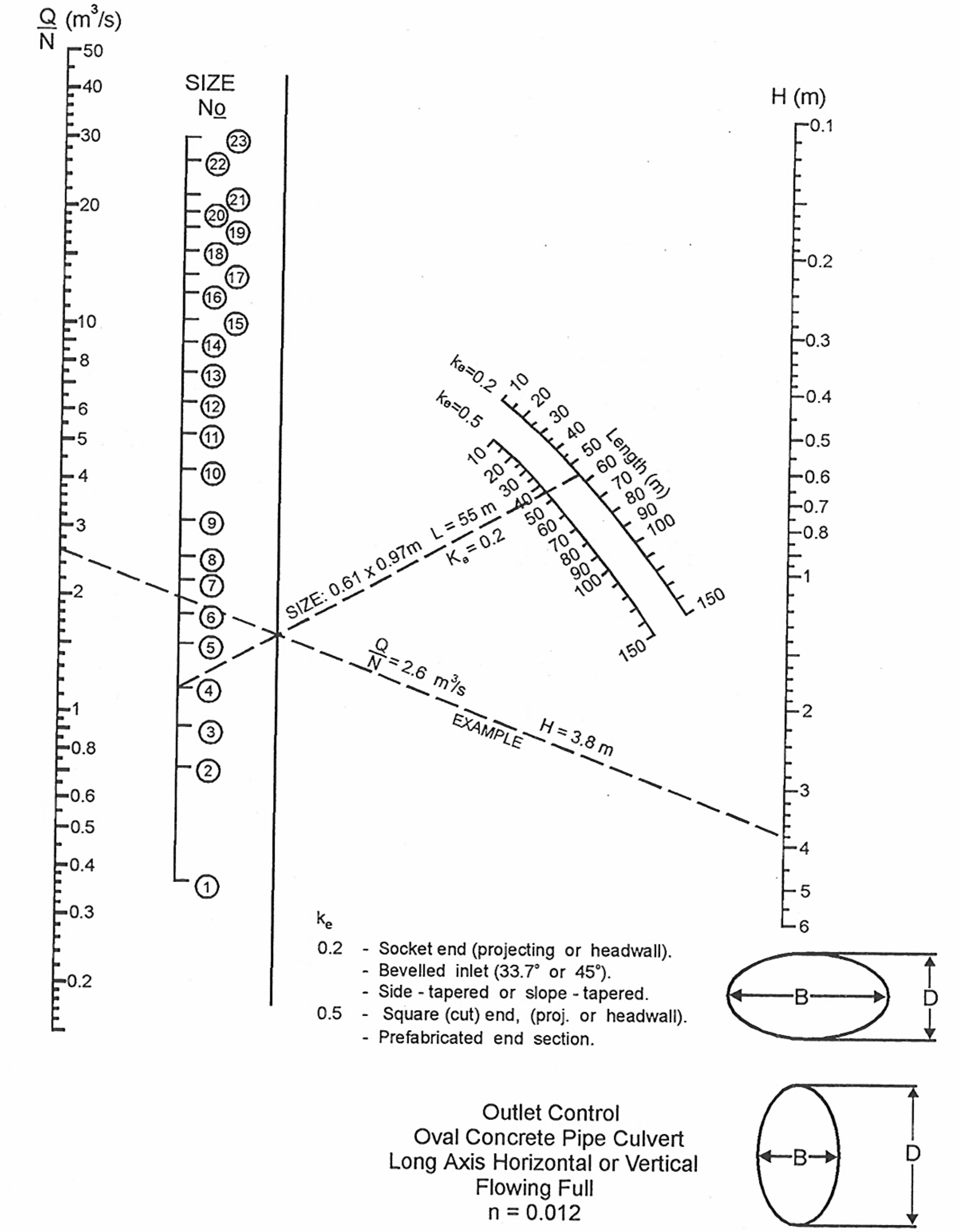
Source: Herr (1977)

Design Chart 5.48: Outlet Control: Pipe Arch SPCSP Culvert - Flowing Full



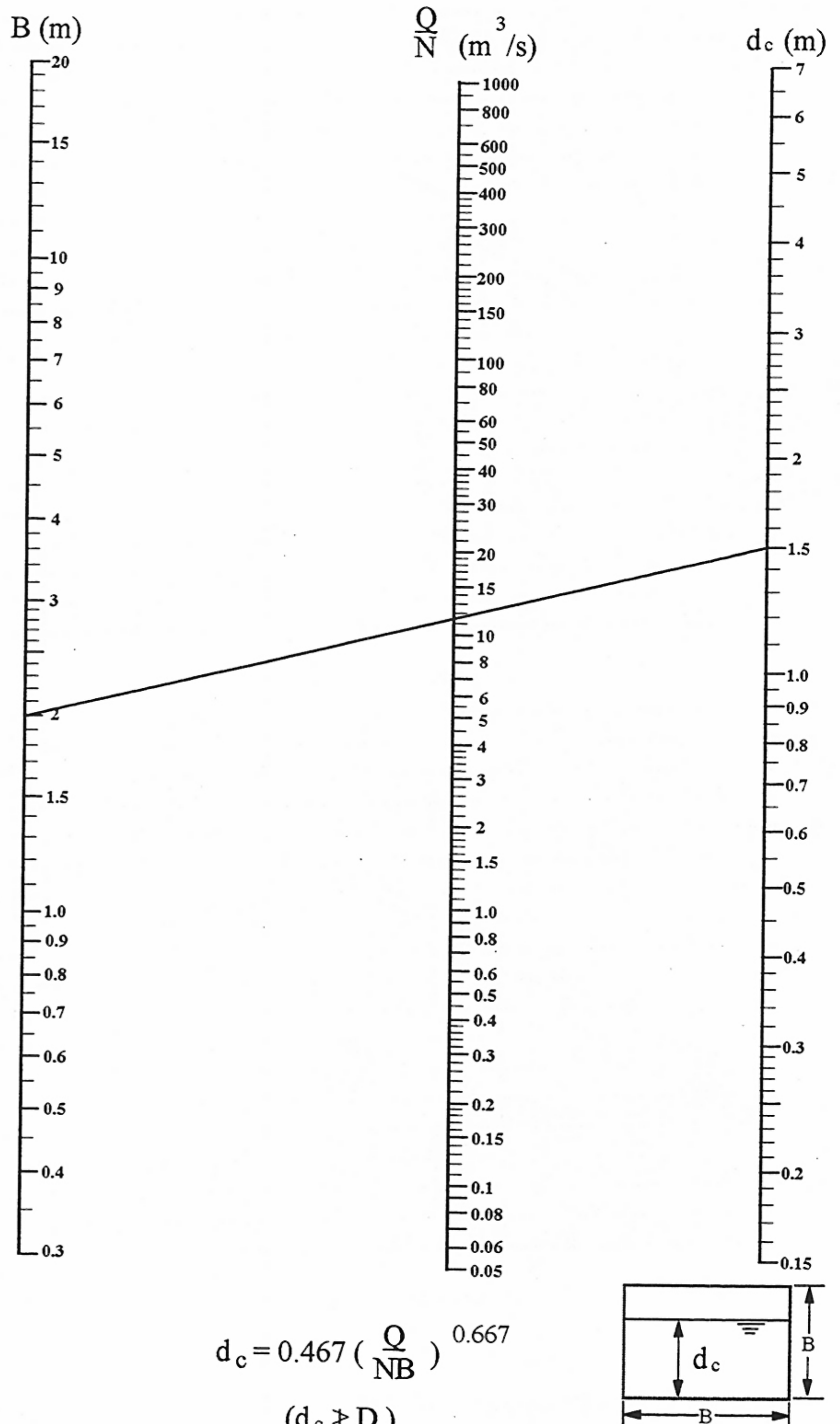
Source: Herr (1977)

Design Chart 5.49: Outlet Control: Elliptical Concrete Culvert



Source: Herr (1977)

Design Chart 5.50: Critical Depth - Rectangular Sections



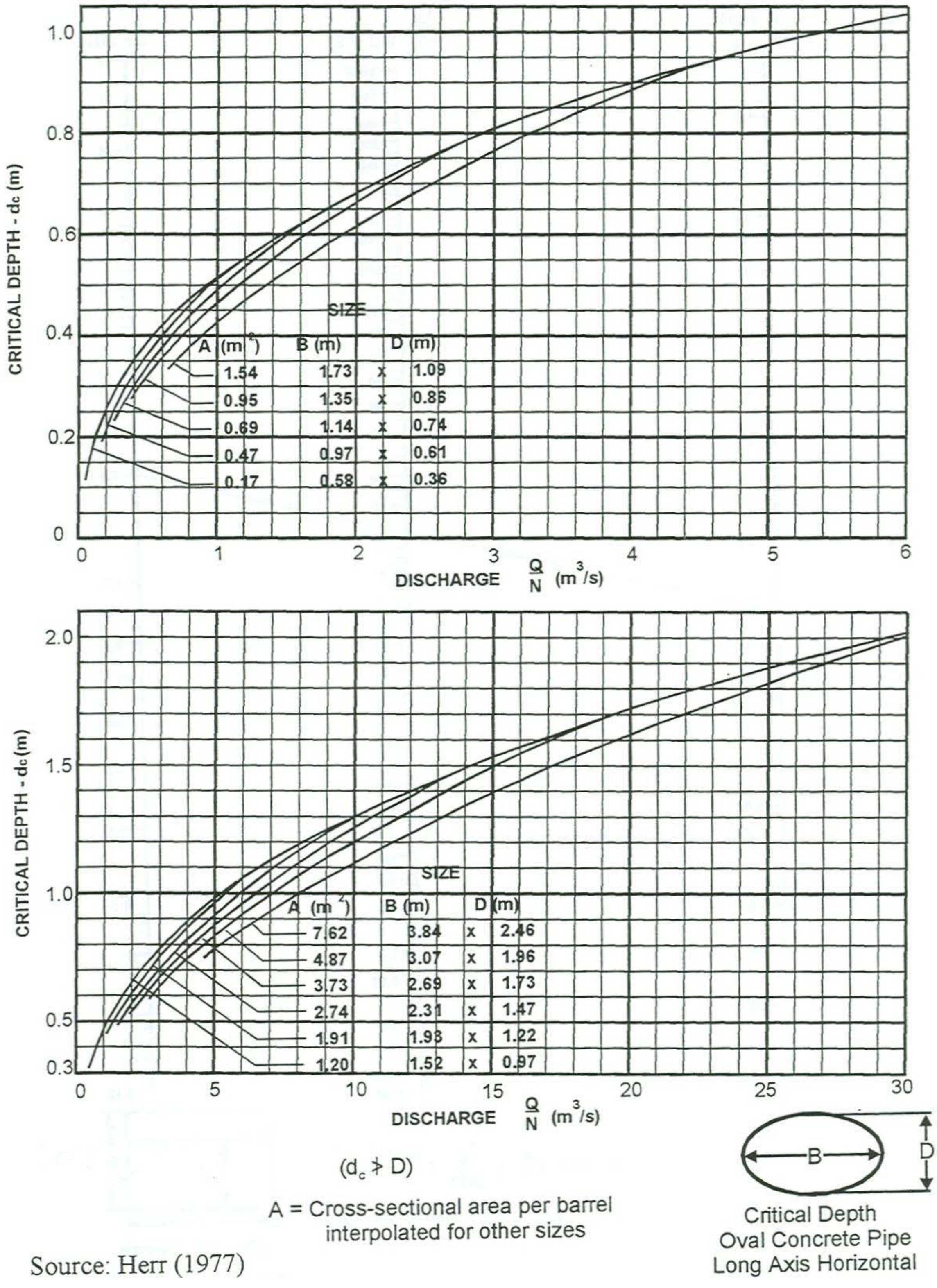
$$d_c = 0.467 \left(\frac{Q}{NB} \right)^{0.667}$$

$$(d_c \not> D)$$

Critical Depth
Rectangular Section

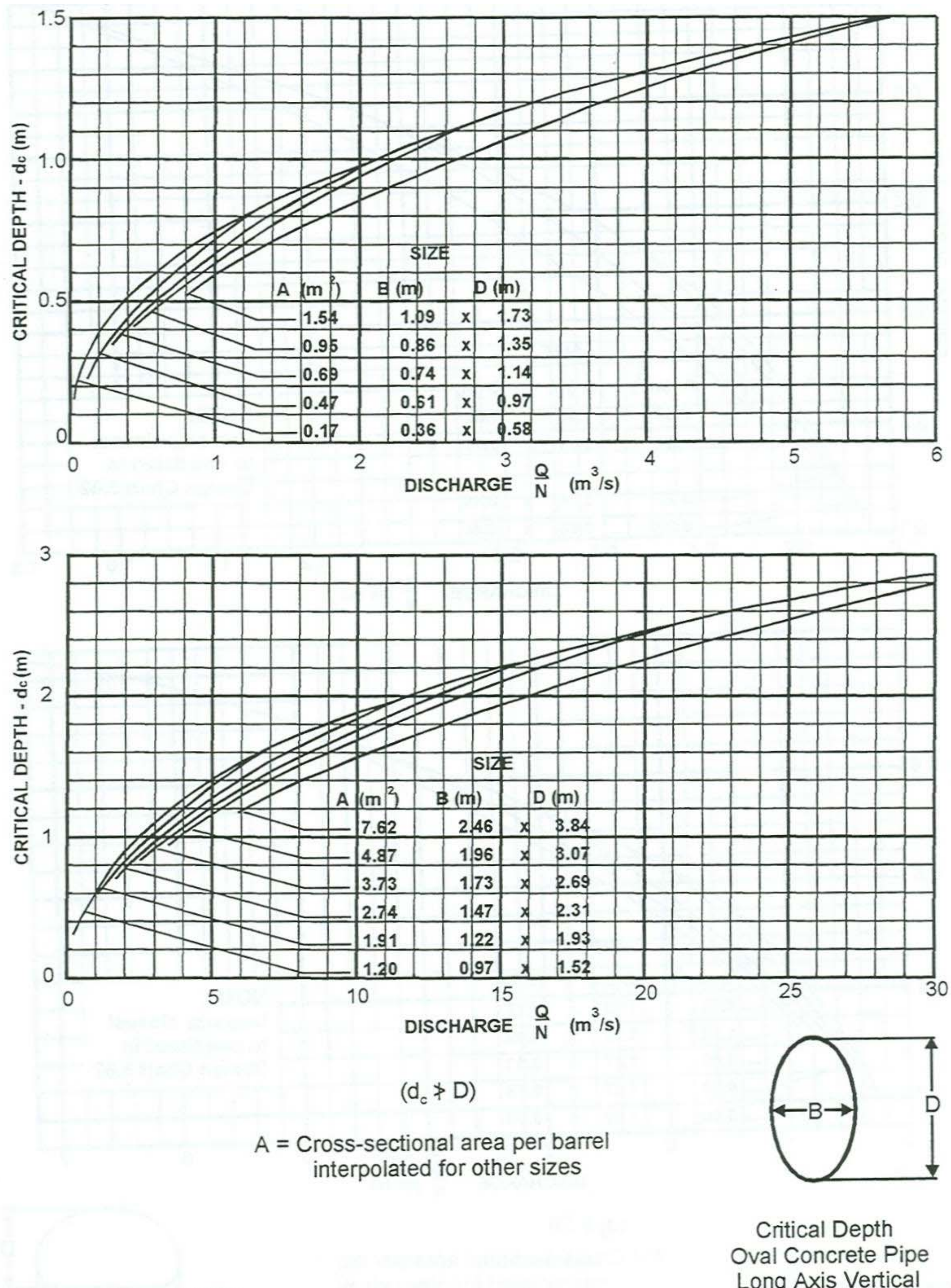
Source: Herr (1977)

Design Chart 5.51: Critical Depth: Horizontal Ellipse Concrete Pipes



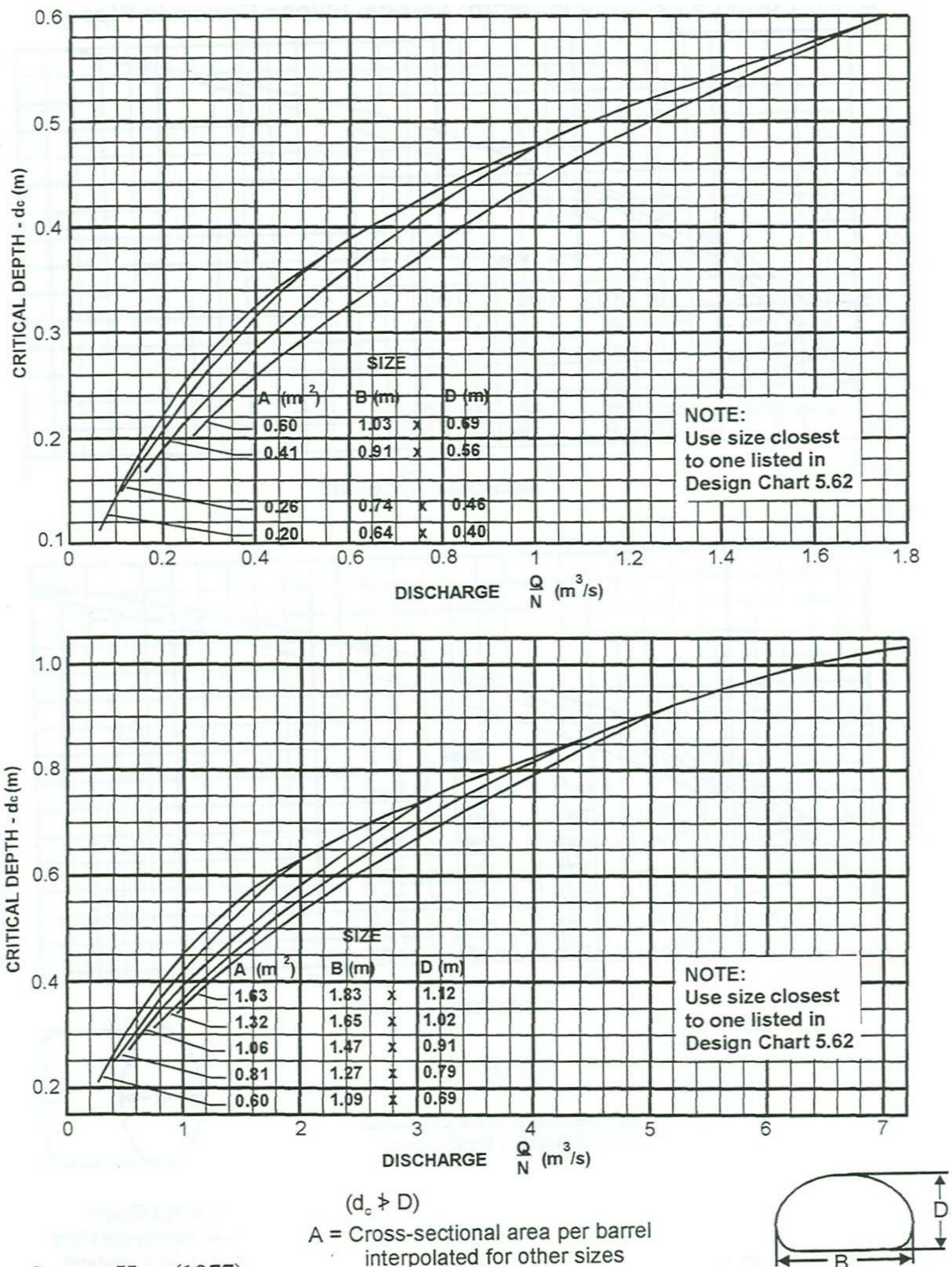
Source: Herr (1977)

Design Chart 5.52: Critical Depth: Vertical Ellipse Concrete Pipes



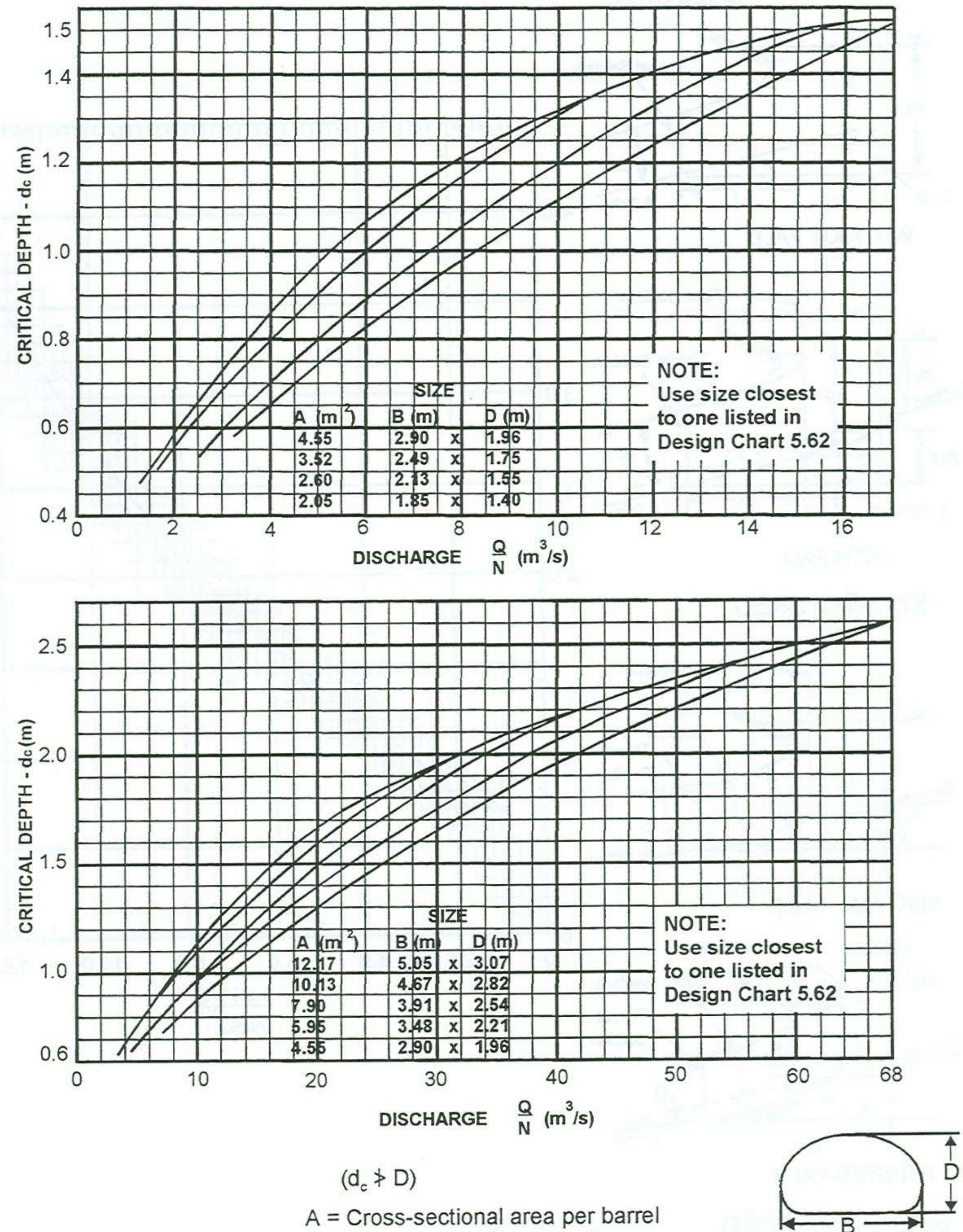
Source: Herr (1977)

Design Chart 5.53: CSP Pipe Arch Culverts



Source: Herr (1977)

Design Chart 5.54: SPCSP Pipe Arch Culverts

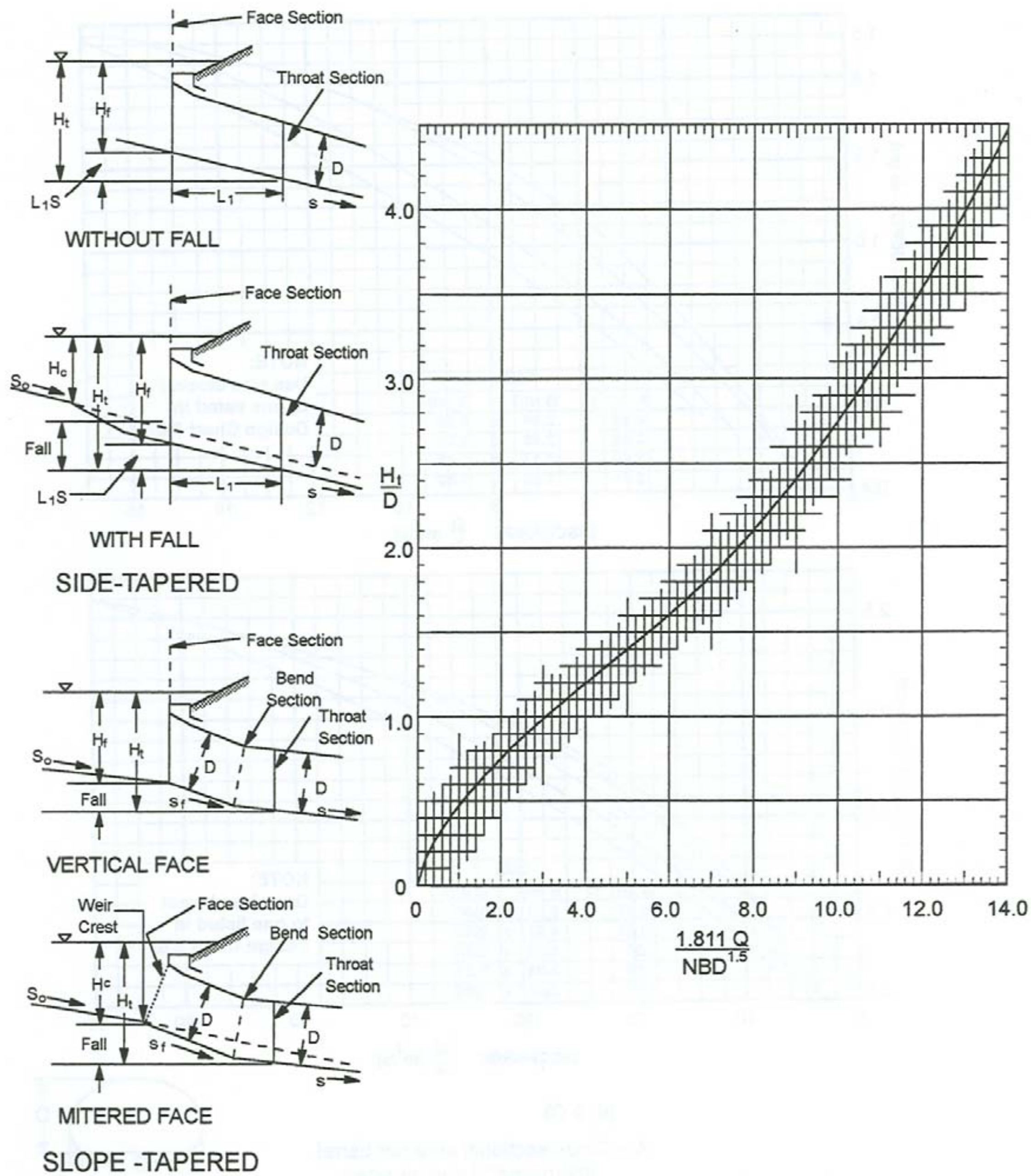


($d_c \not> D$)

A = Cross-sectional area per barrel interpolated for other sizes

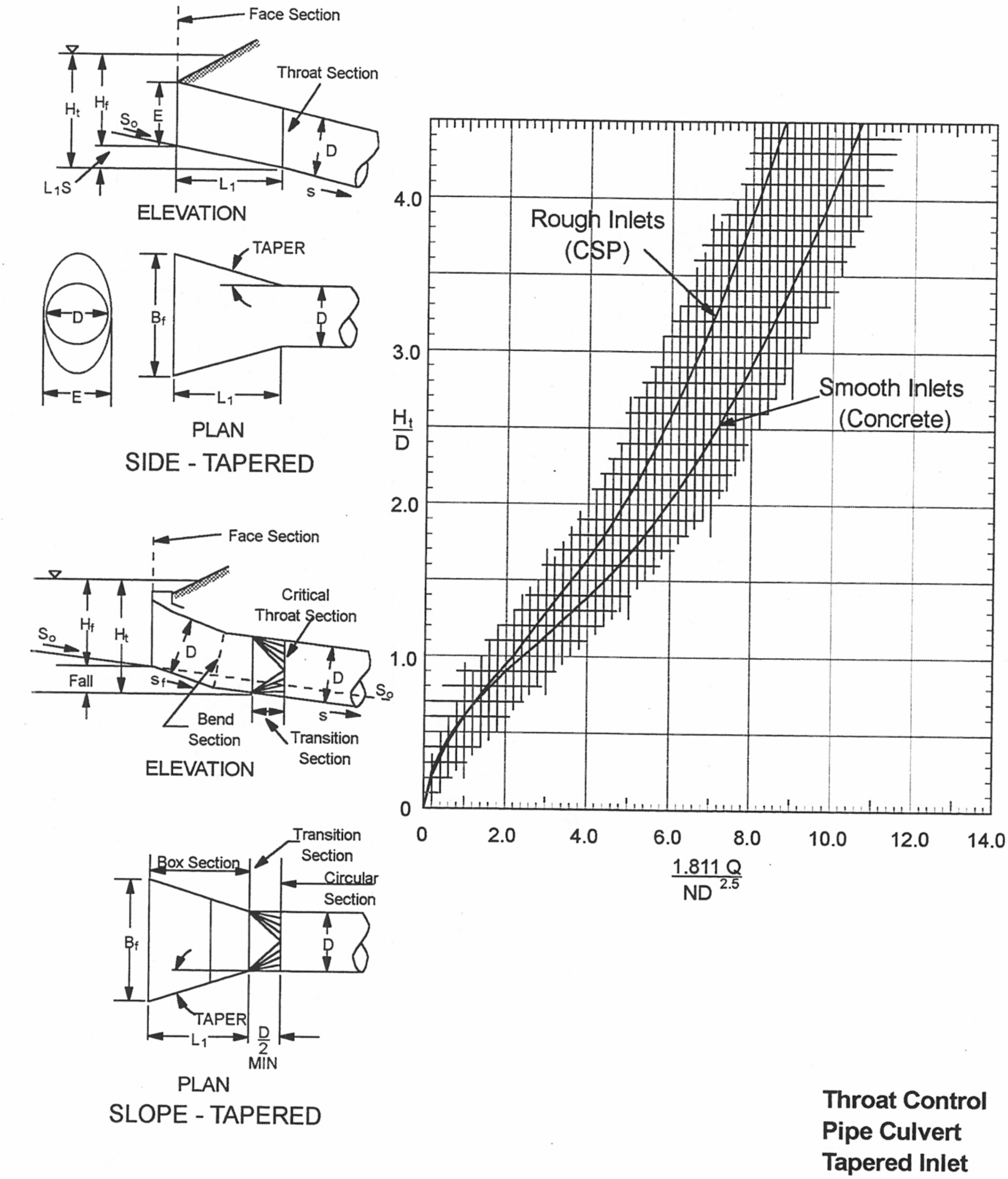
Source: Herr (1977)

Design Chart 5.55: Tapered Inlets: Throat Control - Box Culverts



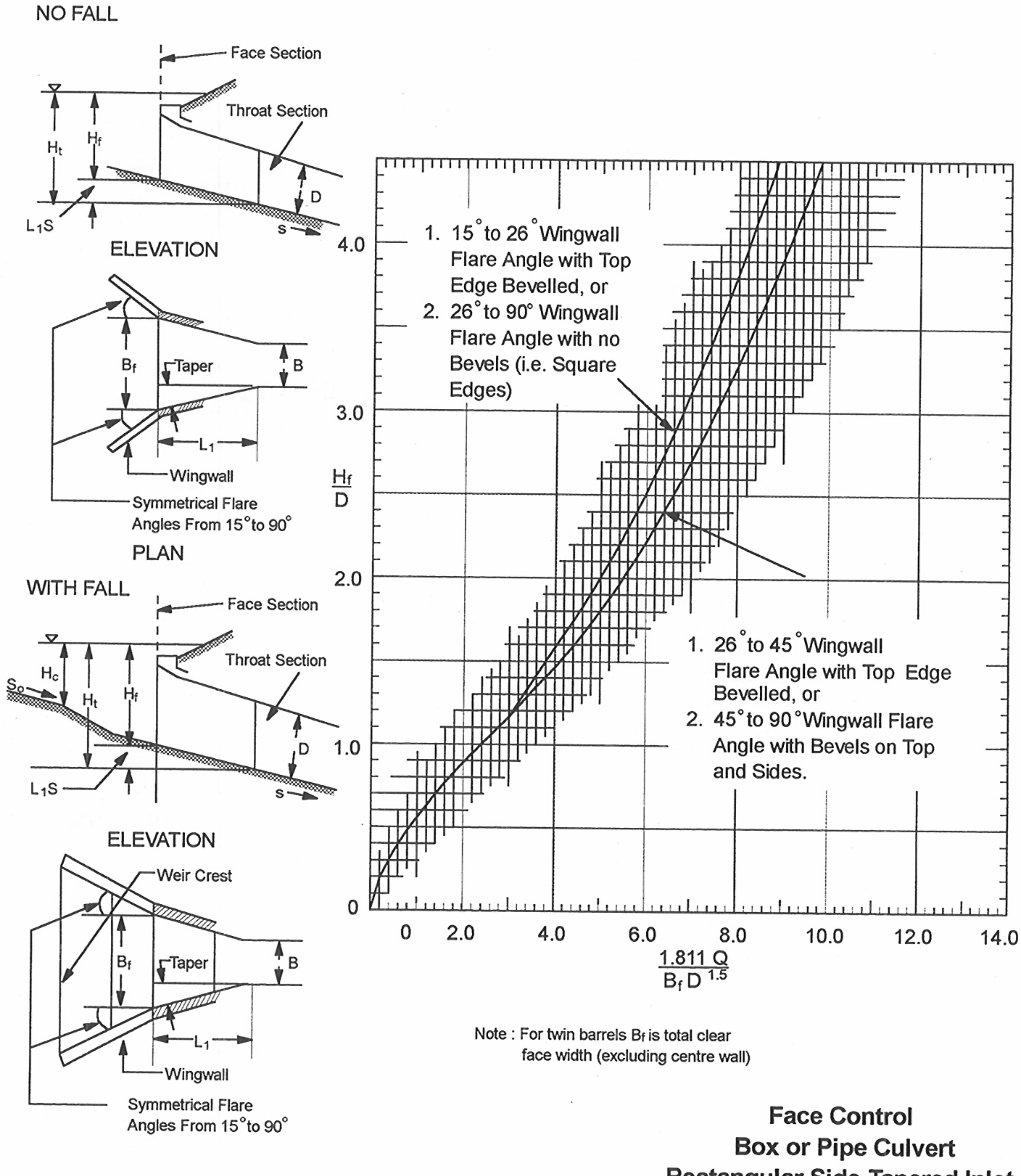
Source: Harrison (1972)

Design Chart 5.56: Tapered Inlets: Throat Control - Circular Culverts



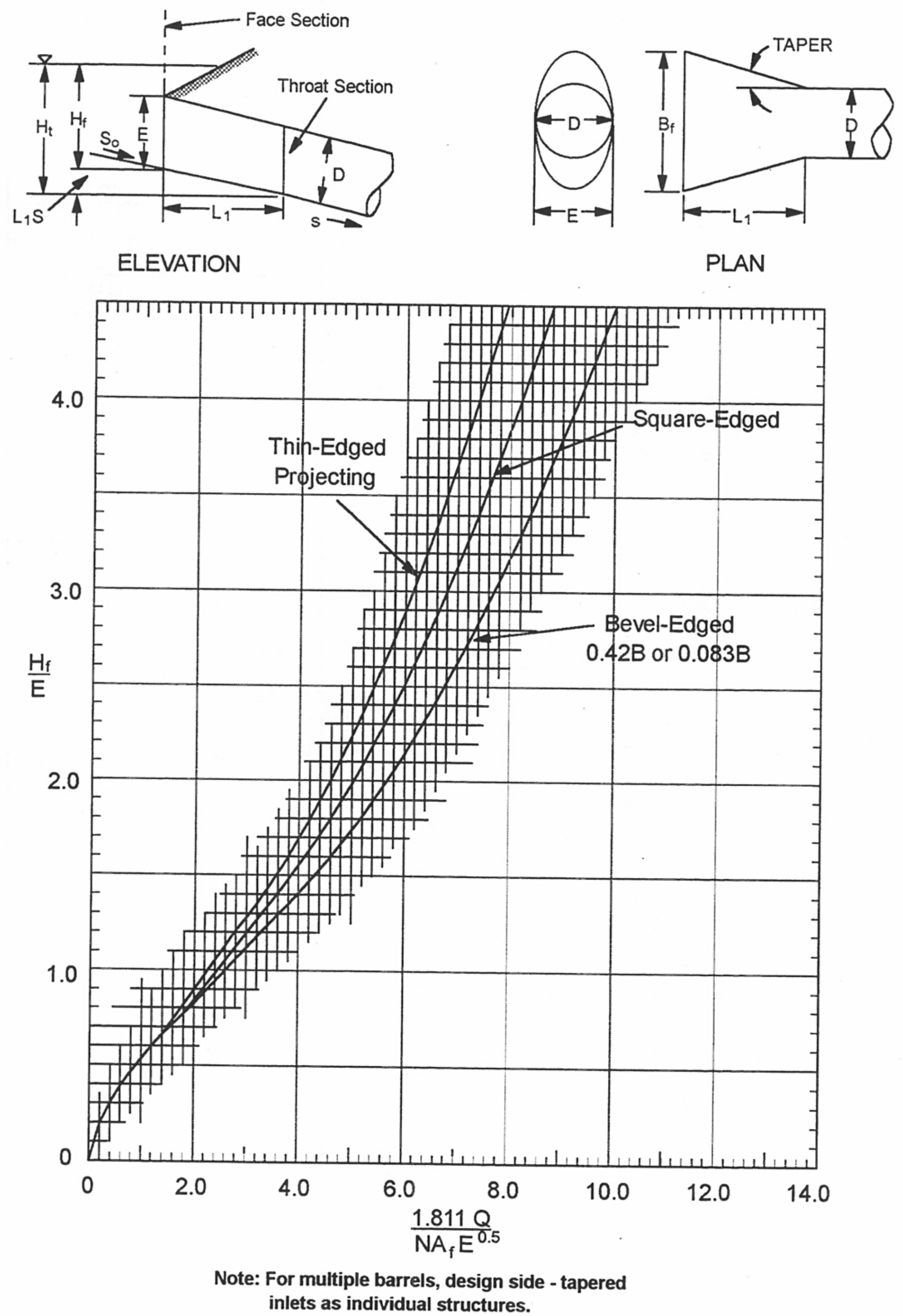
Source: Harrison (1972)

Design Chart 5.57: Side Tapered Inlets: Face Control - Box/Pipe Culverts



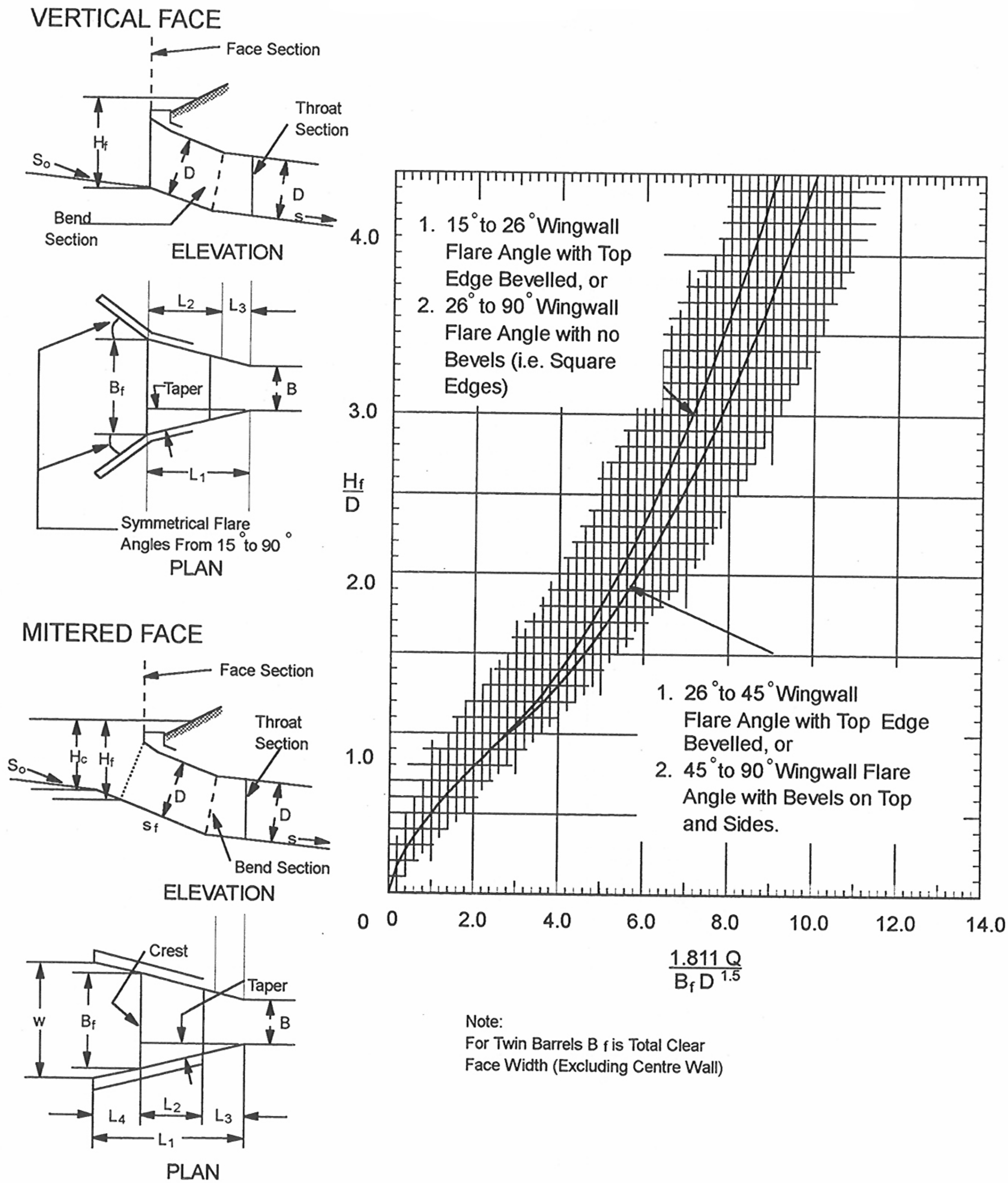
Source: Harrison (1972)

Design Chart 5.58: Side Tapered Inlets: Face Control - Non-Rectangular Pipe Culverts



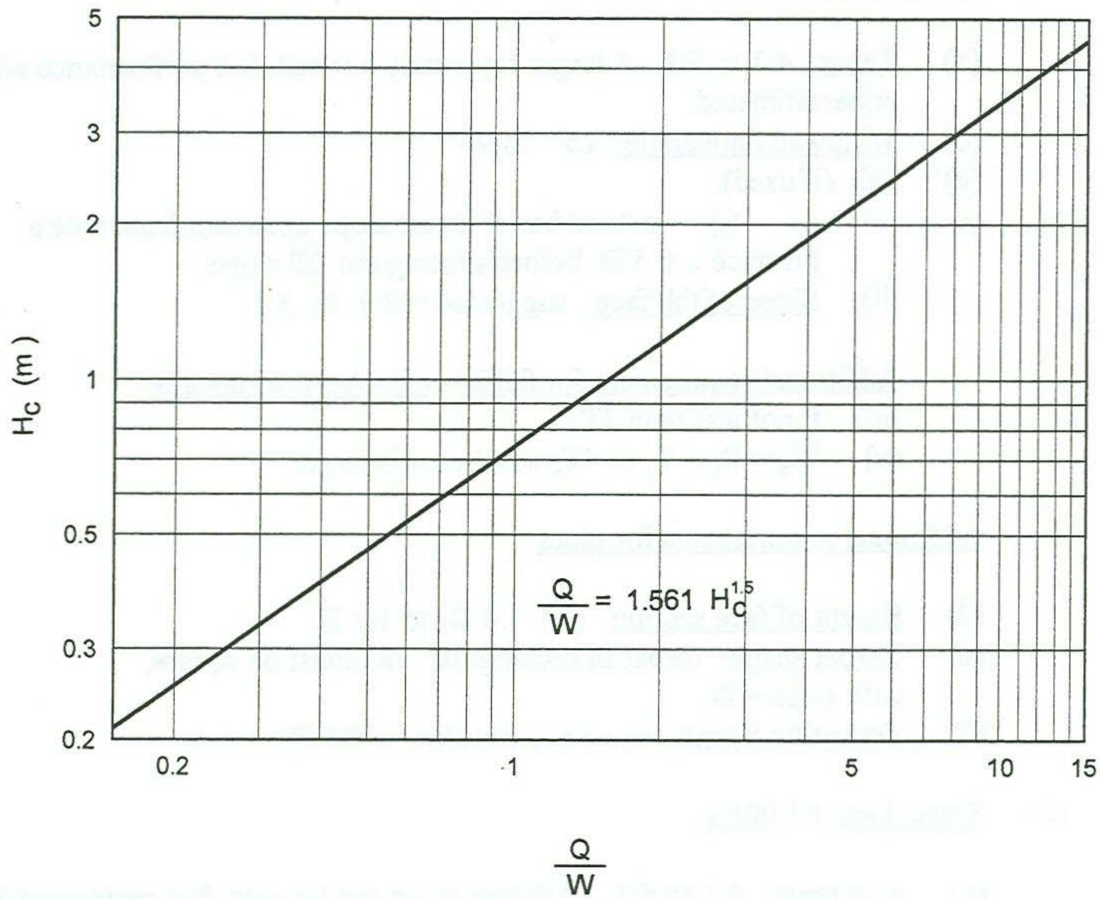
Source: Harrison (1972)

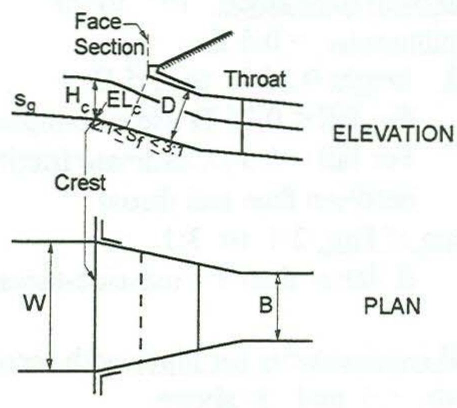
Design Chart 5.59: Rectangular Slope Tapered Inlets: Face Control - Box/Circular Pipe Culverts



Source: Harrison (1972)

Design Chart 5.60: Headwater for Crest Control





Design Chart 5.61: Improved Inlets: Dimensional Requirements

(1) Side-Tapered Inlets

(a) Taper: 4:1 to 6:1. A larger taper may be used, but performance will be underestimated.
(b) Wingwall flare angle: 15E to 90E
(c) Fall (if used):
  (i) extend barrel invert slope upstream from face a distance ∃ 0.5 D, before starting the fall slope.
  (ii) Slope of fall face: suggested = 2:1 to 3:1.

  Additional requirement for fall if not between wingwalls.
  (iv) P not less than 3T.
  (v) $W_p = B_f + T$, or 4T, whichever is larger.

Additional requirements for pipes.

(d) Height of face section (E): 1.0 D to 1.1 D.
(e) Throat shape: throat of rectangular inlet must be square, with sides = D.
(f) Transition length: square to circular: ∃ 0.5 D.

(2) Slope-Tapered Inlets

(a) Side taper: 4:1 to 6:1. A larger taper can be used, but performance will be underestimated.
(b) Wingwall flare angle: 15E to 90E
(c) Minimum L_3 = 0.5 B.
(d) Fall: height 0.25 D to 1.5 D.
  For fall < 0.25 D use side-tapered inlet.
  For fall > 1.5 D, estimate friction losses between face and throat.
(e) Slope of Fall: 2:1 to 3:1
  If flatter than 3:1 use side-tapered inlet.

Additional requirements for pipes with rectangular inlets.
See (1) (d), (e) and (f) above.

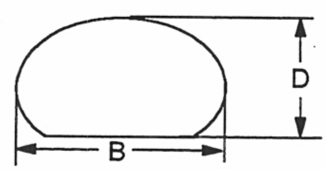
Source: Harrison (1972)

Design Chart 5.62: Dimensions of Corrugated Steel Pipe Arches

CSPA					SPCSPA				
B (m)	D (m)	A (m^2)	p (m)	R (m)	B (m)	D (m)	A (m^2)	p (m)	R (m)
			*					+	
0.56	0.42	0.19	1.57	0.12	2.06	1.52	2.49	5.86	0.42
0.68	0.50	0.27	1.88	0.14	2.24	1.63	2.90	6.34	0.46
0.80	0.58	0.37	2.20	0.17	2.44	1.75	3.36	6.83	0.49
0.91	0.66	0.48	2.51	0.19	2.59	1.88	3.87	7.32	0.53
1.03	0.74	0.61	2.83	0.22	2.69	2.08	4.49	7.81	0.57
1.15	0.82	0.74	3.14	0.24	3.10	1.98	4.83	8.30	0.58
1.39	0.97	1.06	3.77	0.28	3.40	2.01	5.28	8.78	0.60
1.63	1.12	1.44	4.40	0.33	3.73	2.29	6.61	9.76	0.68
1.88	1.26	1.87	5.03	0.37	3.89	2.69	8.29	10.74	0.77
2.13	1.40	2.36	5.65	0.42	4.37	2.87	9.76	11.71	0.83
					4.72	3.07	11.38	12.69	0.90
				**	5.05	3.33	13.24	13.66	0.97
					5.49	3.53	15.10	14.64	1.03
					5.89	3.71	17.07	15.62	1.09
					6.25	3.91	19.18	16.59	1.16
					7.04	4.06	22.48	18.06	1.24
					7.62	4.24	25.27	19.28	1.31

Note: dimensions are shown in metres for direct use in calculations.

* Based on equivalent round diameter.
\+ Based on manufacturers' periphery hole spaces.
** Limit of nomographs.



Source: Metric Standards and Design Data for CSP and SPCSP Products Corrug. Steel Pipe Inst.

(1982)

Design Chart 5.63: Fetch Wind Speed Correction Factor

Fetch Length, km	Correction Factor
0	1.00
1	1.09
2	1.15
3	1.20
5	1.25
10+	1.30

Design Chart 6.01: Soil Erodibility Factors

Soil Texture	Organic Content					
	Under 0.5%		0.5% to 2%		2.1% to 4%	
	K	Sediment Yield	K	Sediment Yield	K	Sediment Yield
Sand	.05	L	.03	L	.02	L
Fine sand	.16	L	.14	L	.10	L
Very fine sand	.42	H	.36	M	.28	M
Loamy sand	.12	L	.10	L	.08	L
Loamy fine sand	.24	M	.20	M	.16	L
Loamy very fine sand	.44	H	.38	M	.30	M
Sandy loam	.27	M	.24	M	.19	L
Fine sandy loam	.35	M	.30	M	.24	M
Very fine sandy loam	.47	H	.41	H	.33	M
Loam	.38	M	.34	M	.29	M
Silt loam	.48	H	.42	H	.33	M
Silt	.60	H	.52	H	.42	H
Sandy clay loam	.27	M	.25	M	.21	M
Clay loam	.28	M	.25	M	.21	M
Silty clay loam	.37	M	.32	M	.26	M
Sandy clay	.14	L	.13	L	.12	L
Silty clay	.25	M	.23	M	.19	L
Clay	0.13 - 0.29 (L to M)					

NOTE:
At critical locations apply Thaw Factor where prolonged surface thawing is expected.

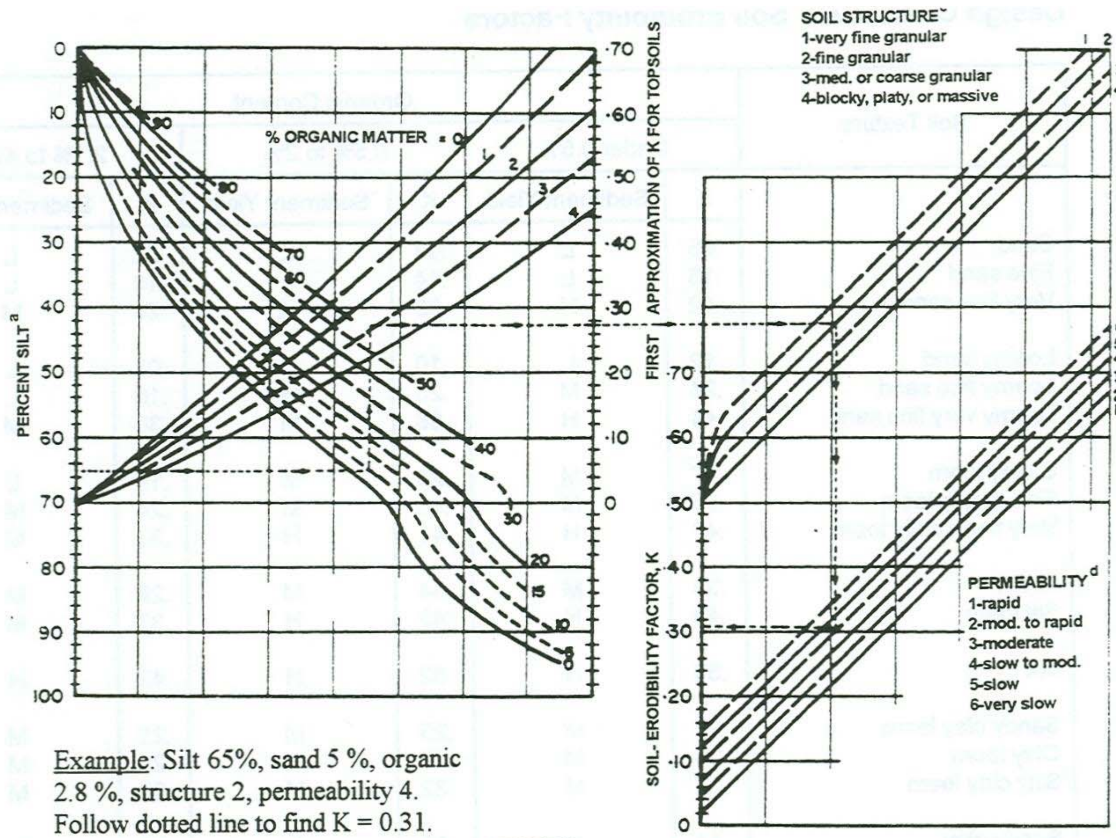
Thaw Factor

Predominantly sand soils	1.3
Predominantly loam soils	1.1
Predominantly clay soils	1.4

At critical locaions, for periods in which prolonged surface thawing is expected, multiply K values by the above factors.

Source: University of Guelph (various years)
Dickinson, et. Al (1982)

Design Chart 6.02: Wischmeier Nomograph



Example: Silt 65%, sand 5 %, organic 2.8 %, structure 2, permeability 4. Follow dotted line to find K = 0.31.

NOTES

a. Silt defined as 2 um to 75 um in MTO soil classification system.

b. Sand was originally defined as 0.10 mm to 2.00 mm in Wischmeier's nomograph.

c. Soil Structure Equivalents

1.	Very fine granular	Very fine sand
2.	Fine granular	Fine or medium sand
3.	Medium or coarse granular	Coarse sand or gravel
4.	Blocky, platey or massive	Clay

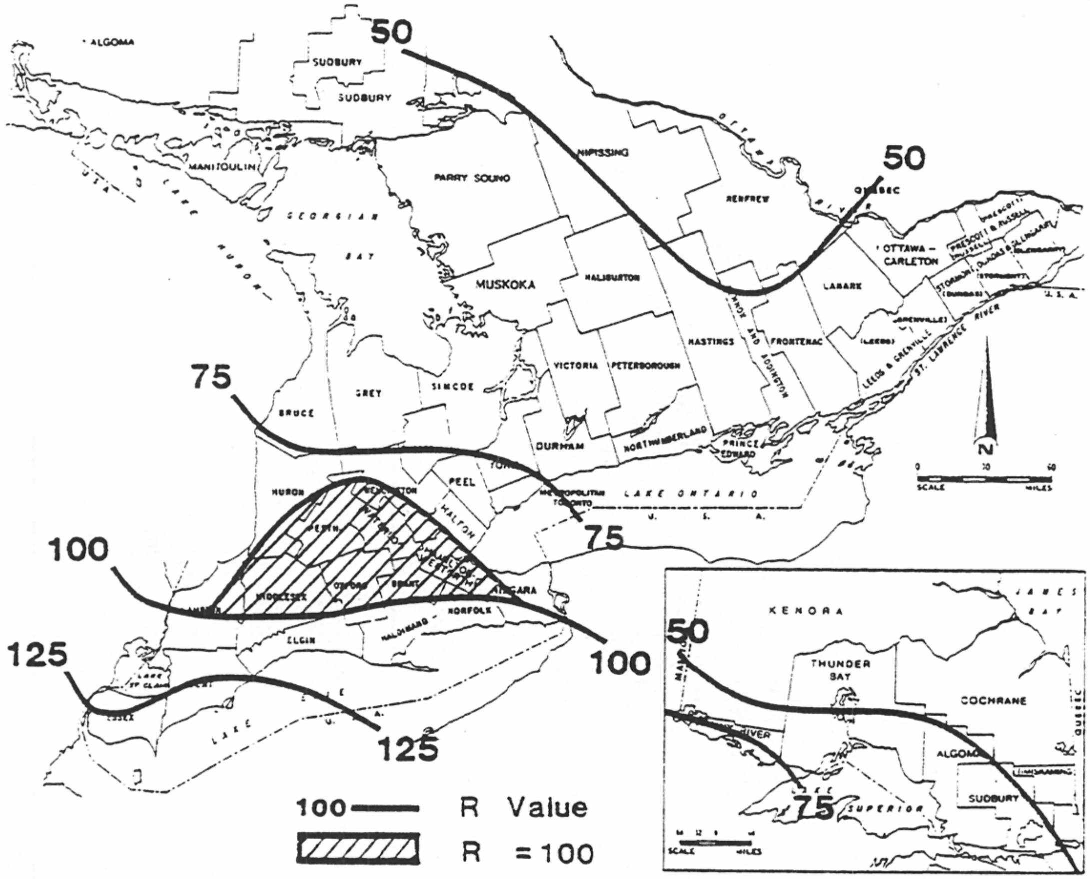
d. Permeability Equivalents

1.	Rapid	Gravel
2.	Rapid to moderate	Sand
3.	Moderate	Sandy loam
4.	Moderate to slow	Clay loam
5.	Slow	Silty clay
6.	Very slow	Light to heavy clay

Source: Wischmeier, W.H. et al. (1971)

Source: Wischmeier, W.H. e al. (1971)

Design Chart 6.03: Average Rainfall Factors for Ontario



Dickenson and Wall, University of Guelph and Ontario Pedological Institute (Pers. comm.)

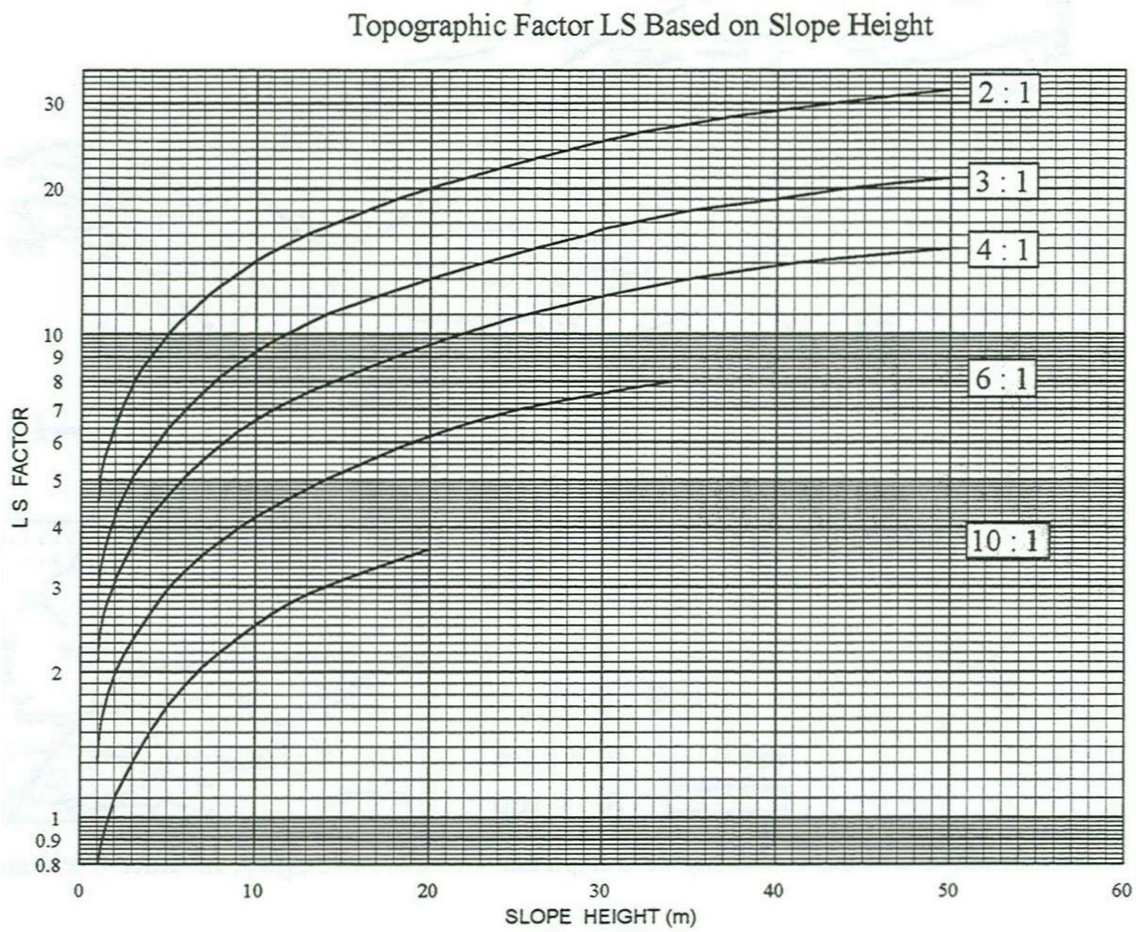
Note:
1. R Values include an allowance for snowmelt
2. See discussion on R value in Chapter 8 for units

Monthly Distribution of Annual R Factor

January	2%	July	19%
February	2%	August	18%
March	3%	September	11%
April	6%	October	7%
May	8%	November	6%
June	15%	December	3%

Note: Values include an allowance for snowmelt.

Design Chart 6.04: Topographic Factors



Source: Israelson (1980)

Design Chart 6.05: Erosion Control Factors

Typical VM Factors*

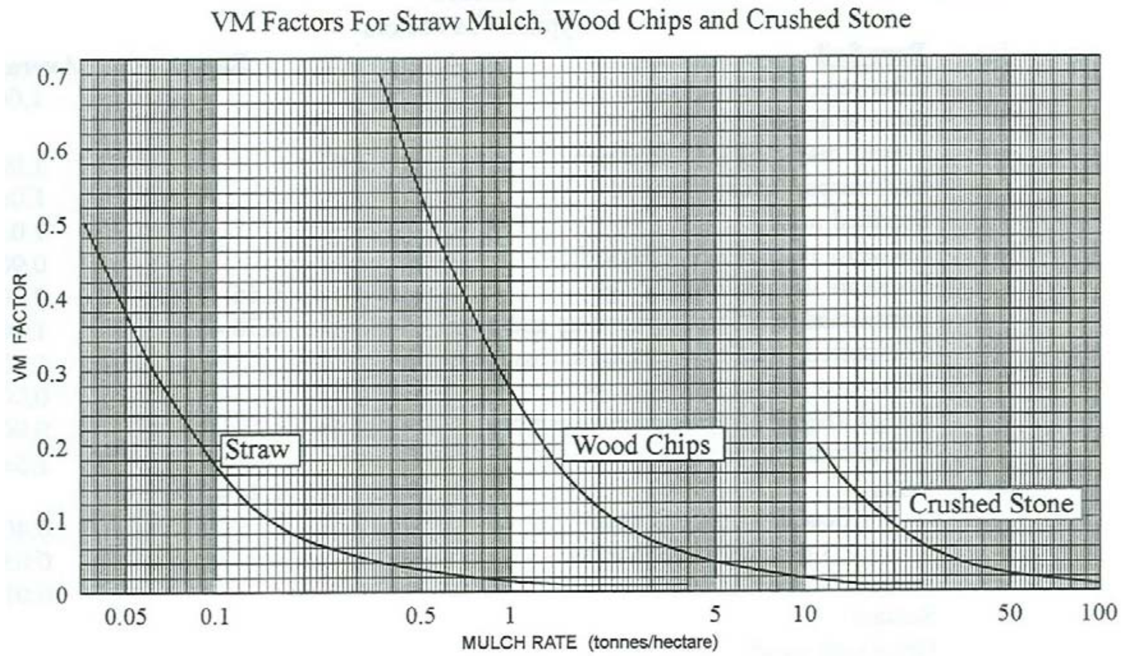
		Range	Average
1.	**Bare Soil**		
	After removal of root zone (normal design value)		1.00
	Freshly disced to 150 - 200 mm		1.00
	Undisturbed, except scraped		1.0±
	Scarified only		1.0±
	Loose to 300 mm deep, smooth		0.90
	Loose to 300 mm deep, rough	0.66 - 1.30	0.80
	Compacted, bulldozer scraped up and down	0.76 - 1.31	1.30
	Compacted, bulldozer scraped across slope		1.20
	Ditto, except root raked across		0.90
	Rough irregular tracked in all directions		0.90
	Compacted fill		1.5±
2.	Grass	1.24 - 1.71	
	Seeded - up to 60 days**		0.40
	- 60 days to 12 months		0.05
	- after 12 months		0.01
	Sodded		
	Grass with weeds		
		See next page	

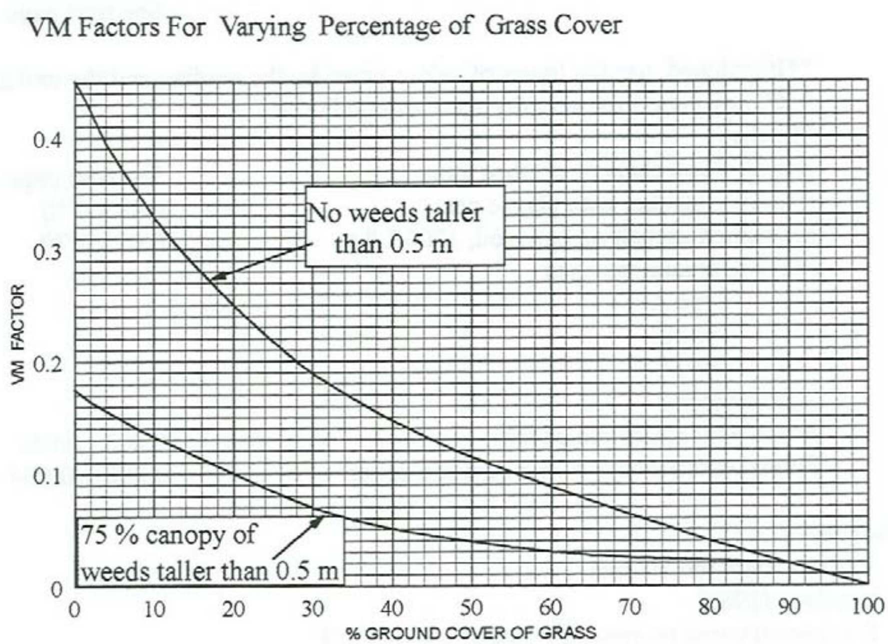
**If mulched, use the lower of values given by the seeding and the mulch. Extend the 60 day period for very dry weather conditions.

		Range	Average
3.	Mulches		
	Straw, wood chips or crushed stone	See next page	
	Excelsior blanket with plastic net	0.04 - 0.10	0.07
	Emulsified asphalt on bare soil, 1700 L/ha	0.65 - 0.70	0.68
	MTO hydraulic mulches:		
	Type A		
	Type B		
	Type C		
4.	Miscellaneous		
	Brush > 80 % ground cover	0.003 - 0.040	0.02
	Forest, undisturbed, > 75 % ground cover	0.0001 -0.004	0.002

Sources:
1. Various Research Studies
2. Israelson (1980)
3. Provisional based on research sponsored by MTO

Design Chart 6.05: (Continued) : Erosion Control Factors





Source: Israelson (1980)

Design Chart 6.06: Provisional Sediment Delivery Ratio for Sheet Flow

Type of Surface	Sheet Flow Travel Distance				
	Up to 3m	3 to 20 m	21 to 50 m	51 to 100 m	over 100 m
Predominantly clay soils	1.0	1.0	1.0	0.8	0.4
Predominantly loam soils	1.0	1.0	0.8	0.4	0.2
Predominantly sand/gravel	1.0	0.8	0.4	0.2	0.0
Heavily vegetated buffer	1.0	0.8	0.4	0	0

Notes:

1. Exclude paved surface from travel distance
2. To be used as a rough guide only; if in doupt use the higher value
3. Reference, Dickinson and Wall, University of Guelph and Ontario Pedological Institute (personal Communications)

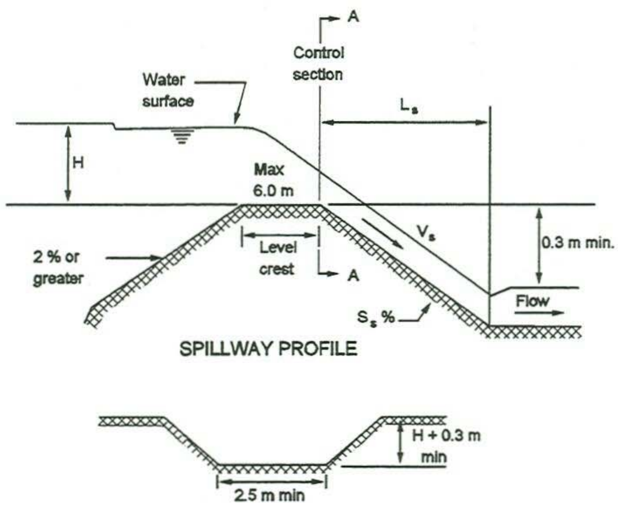
Design Chart 6.07: Design Data for Emergency Spillways (Sheet 1 of 2)

Head H (m)	Spillway Variables	Bottom Width b (m)								
		2.5	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0
0.2	Q m3/s	0.28	0.33	0.33	0.64	0.64	0.75	0.86	0.97	1.08
	S_s %	3.6	3.6	3.5	3.5	3.5	3.5	3.5	3.5	3.5
	Vs m/s	0.9	0.95	0.97	0.97	0.97	0.97	0.97	0.97	0.97
	L_s m	12	12	12	12	12	12	12	12	12
0.3	Q	0.54	0.65	0.86	1.06	1.26	1.47	1.68	1.83	2.03
	S_s	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
	Vs	1.21	1.21	1.21	1.21	1.21	1.21	1.21	1.21	1.21
	L_s	15	15	15	16	15	15	16	16	16
0.4	Q	0.92	1.08	1.43	1.71	2.06	2.38	2.64	2.99	3.29
	S_s	2.8	2.8	2.7	2.7	2.7	2.7	2.7	2.7	2.7
	Vs	1.41	1.41	1.41	1.41	1.44	1.44	1.44	1.44	1.44
	L_s	18	18	19	19	19	19	19	20	20
0.5	Q	1.40	1.63	2.08	2.55	3.03	3.46	3.92	4.37	4.79
	S_s	2.6	2.6	2.6	2.5	2.5	2.5	2.5	2.5	2.5
	Vs	1.56	1.56	1.57	1.58	1.60	1.62	1.62	1.62	1.62
	L_s	23	23	23	23	24	24	24	24	24
0.6	Q	1.94	2.26	2.89	3.41	4.16	4.73	5.35	5.49	6.55
	S_s	2.4	2.4	2.4	2.4	2.4	2.3	2.3	2.3	2.3
	Vs	1.72	1.72	1.72	1.72	1.76	1.73	1.76	1.78	1.78
	L_s	27	27	27	27	28	28	28	28	28
0.7	Q	2.63	3.02	3.77	4.58	5.39	6.14	6.93	7.67	8.43
	S_s	2.4	2.4	2.3	2.3	2.3	2.3	2.2	2.2	2.2
	Vs	1.86	1.86	1.86	1.89	1.89	1.91	1.92	1.92	1.92
	L_s	31	31	31	31	32	32	32	32	32

Notes:

- See sheet 2 for sketch and legend
- Table is correct for n=0.04 (grass). For use with other values of n see sheet 2
- Table may be used for spillway lengths (parallel to flow) of 0.6 m or less.
- Spillway sections to right of heavy lines should be used with caution, as they may be poorly proportioned.
- Table is based in the reference *Design of Sedimentation Basins*, U.S Transporation Reseach Board (1980)

Design Chart 6.07: Design Data for Emergency Spillways (Sheet 2 of 2)



1. For spillway n values other than 0.04 (grass), use adjustmed Q.
   Adjusted Q = Design Q x Adjusment factor given below.

n	Factor
0.02	0.08
0.03	0.87
0.06	1.18

2. Values of H, S_s, V_s and L_s for givern values of Q (adjusted if necessary) and b are given on sheet 1.

   S_s is minimum slope of spillway downstream from crest.
   L_s is minimum length of spillway downstream from crest
   V_s given bt sheet 1 corresponds to the minimum calue of S_s. If a slope S steeper than S_s is used, calculate corresponding V as follows:

   $V = V_v \, (S / S_s)^{0.3}$ m/s
3. Maximum velocity for grass-lined spillway ≈ 1.8 m/s

AASHTO   Acronym for American Association of State Highway and Transportation Officials. (AA)

Abutment   A wall supporting the end of a bridge or span, and sustaining the pressure of the abutting earth. (MTC)

Aggradation   Progressive raising of the general level of a channel bed over a period of years by an accumulation of sediment. (MTC)

Allowable Fish Passage Velocity   The maximum velocity fish can tolerate when passing upstream through a culvert. (MTC)

Alluvial Stream   A stream whose channel is formed in erodible soil deposited by flowing water and which carries sediment loads of similar material. (MTC)

AMC   See Antecedent Moisture Condition.

Angle of Repose   The maximum angle, as measured from the horizontal, at which unsupported granular particles can stand. (ASCE)

Antecedent Moisture Condition   The degree of wetness of a watershed's surface soils at the beginning of a storm. (AA)

Approach Velocity   The mean velocity in a conduit or channel immediately upstream of a weir, dam, throat, culvert, bridge or structure. (MTC)

Apron   Protective material laid on a stream bed to prevent scour at a bridge pier, abutment, culvert outlet, toe of a slope or similar location. (MTC)

Area of Scour   The cross sectional area of material scoured from the stream bed, measured below the natural bed. (MTC)

Armour   Artificial surfacing of bed, banks, shores or embankments to resist scour or erosion. (MTC)

Articulated Concrete Mattress   Concrete slabs or blocks joined together with corrosion-resistant fasteners. The mattress can move without separating as scour occurs in the underlying soil. (AA)

Average Daily Traffic (ADT)   An average value for the daily vehicular traffic on a specific roadway. (FHWA)

Backwater   The upstream increase of water level caused by the presence of a bridge, culvert or other constriction or obstruction. (MTC)

Bankful Discharge   Discharge that fills a channel to the point of overflowing. (AA)

Bed Load   Sand, silt, gravel, rock or other mineral matter which is carried by a stream on or immediately above its bed. (MTC)

Berm   A level shelf interrupting the continuity of a slope. (MTC)

Bernoulli's Equation   A theory advanced by Daniel Bernoulli that the energy head at any section in a channel is equal to the energy head at any other downstream section plus the intervening energy losses. (AA)

Best Management Practice   The term refers to water quality facilities such as extended detention and wet ponds, infiltration systems, constructed wetlands and various vegetative practices. (MOEb)

Bevel   A sloping surface formed on the edge(s) of a culvert inlet to improve its hydraulic efficiency. (MTC)

Bioengineering   See Soil Bioengineering.

Biota   The combined fauna and flora of a geographical area or geological period. (MOEa)

Braided Stream   A relatively wide and shallow stream having numerous unstable interlacing channels separated by gravel or sand bars and small unstable islands. (MTC)

Bridge Waterway   That part of a bridge opening that is or may be occupied by water. (MTC)

Caisson   A permanent shell within which excavation is carried out and which sinks under its own or added weight until it reaches firm ground. (MTC)

Canadian Shield   Within Ontario, a large (650 000 km^2) area of exposed or shallowly buried Precambrian bedrock occupying much of the central and northern portions of the province. The Shield terrain in Ontario is typically forest-covered, with innumerable rock knobs, hills, valleys, lakes, wetlands and beaver ponds. (MTC)

Catchbasin   A basin of concrete or other material, covered by a grate, and located in a gutter or ditch to intercept stormwater for transmission to a sewer or other outlet. (MTC)

Channel Diversion   A channel with a supporting ridge on the lower side constructed across the slope to divert water from areas where it is in excess to sites where it can be used or disposed of safely. (MTC)

Channel Routing   See Flood Routing.

Check Dam   A low dam constructed in a ditch or other channel to reduce the gradient, decrease the flow velocity, minimize erosion and in some cases promote the deposition of sediment. (MTC)

Check Flood   A flood used to ensure that a bridge designed for the normal design flood will withstand flood without structural failure. (MTC)

Chute   A steeply inclined open or closed conduit for conveying water from a higher to a lower level. (MTC)

Cofferdam   A temporary earth dike, wall or other structure built around a site to permit construction in the dry. It may be an earth dam or a row or rows of sheet piling. (ASCE)

Cohesive Soil   A soil that when unconfined has considerable strength when air-dried, and significant cohesion when submerged. (MTC)

Cold Water Fishery   A fresh water, mixed fish population, including some salmonoids. (MOEa)

Combination Inlet   A storm sewer inlet comprising a grate and curb inlet combined in one unit. (MTC)
Common Law   As distinguished from "Roman" or "Civil" law, the body of unwritten law, especially of England, based on longstanding usages and customs and the court decisions and decrees recognizing, affirming and enforcing such usages and customs. (MTC)

Competent Velocity   The velocity of water that can just move a specified size or type of material in a stream bed. (MTC)

Confluence   A junction of two or more streams. (MTC)

Conjugate Depth   The alternate depth of flow involved with the hydraulic jump; i.e. the depths d_1 and d_2 before and after a hydraulic jump. Unlike the alternate depths for a given specific head, the conjugate depths for a hydraulic jump reflect the energy loss from the hydraulic jump. (AA)

Continuity Equation   An equation that states that in steady flow, the discharge at one section of a channel is equal to the discharge at another section if there is no inflow and outflow between the two sections. (MTC)

Control Section   A natural or constructed feature in an open channel which determines the stage-discharge relationship at that point. (MTC)

Conventional Culvert   A closed invert culvert having no major inlet improvement such as a side-tapered or slope-tapered inlet. It may incorporate minor improvements such as bevelled edges, wingwalls, a fall, or a prefabricated end section. (MTC)

Conveyance   A measure of flow capacity of a channel or stream. (MTC)

Crest   The maximum elevation of a flood at a specific location. Other definitions are: 1. The top of dam, dike, spillway, or weir; 2. Overflow portion of a road or embankment; 3. Summit of a wave; 4. Peak of a flood. (AA)

Critical Depth   The depth at which, for a given energy content of the water in a channel, maximum discharge occurs; or the depth at which in a given channel a given quantity of water flows with minimum content of energy. (AA)

Critical Flow   The maximum discharge of a conduit which has a free outlet and has the water ponded at the inlet. (ASCE)

Critical Shear Stress   The minimum shear stress exerted by the flow in a channel required to initiate motion of the particles of the lining material of the channel. (AA)

Critical Velocity   The mean velocity of flow at critical depth. (AA)

Cross Section   A vertical section (profile) of the surface, the ground, and/or underlying material, taken at right angles of the centre line or across an object (a stream, a dam, a structure, etc.) (ASCE)

Cross Fall (Cross Slope)   A transverse slope provided on a pavement or other surface for drainage purposes. (MTC)

Crown   The highest point of the interior of a pipe at a given cross section. (MTC)

Culvert   A conduit, usually covered by fill, whose primary function is to convey surface water through an embankment. (MTC)

Culvert Uplift   The upward movement of a culvert end resulting from buoyancy forces. (MTC)

Curb and Gutter Section   A roadway gutter section in which the curb and gutter are integral. (MTC)

Curve Number   A number between 0 and 100 which indicates the runoff-producing potential of a soil/land use combination when the ground is not frozen. (MTC)

Cutoff   1. A vertical wall below the end of a culvert or other structure. (MTC) 2. A natural or artificial channel that shortens the length of a stream; natural cutoffs may occur across the neck of a meander loop. (AA)

Debris   Any material transported by the stream, either floating or submerged, such as logs, brush, suspended sediment, bed load, or trash that may lodge against a structure. (AA)

Degradation   The progressive lowering of the general level of a stream channel over a period of years by erosion. (MTC)

Deposition   The dropping of material as a result of a reduction of velocity of the transporting agent (wind or water). (MTC)

Design Flood   The maximum rate of flow for which a drainage facility is designed and thus expected to accommodate without exceeding the adopted design constraints. (AA)

Design Storm   Selected storm of a given frequency used for designing a storm drainage system. Also a hypothetical storm derived from intensity-duration-frequency curves by reading the rainfall intensity from these curves for various durations for the frequency of interest and rearranging these rainfall intensities to fit an assumed storm pattern and storm duration. (AA)

Detention Pond   A pond for temporarily storing surface runoff and releasing the stored water at a controlled rate. (MTC)

Dike   An embankment or wall, usually along a watercourse or flood plain, to prevent overflow on to adjacent low land. (MTC)

Direct Runoff   Runoff from surface flow or rapid subsurface flow which enters a stream channel during or soon after a storm. (MTC)

Dissolved Oxygen (DO)   The oxygen dissolved in a liquid. (FHWA)

Ditch   A small artificial drainage channel having a definite bed and banks. (MTC)

Ditch Inlet   A stormwater inlet to intercept flow in a ditch or swale for transmission to a sewer or other outlet. (MTC)

Diversion   See Channel Diversion.

Drawdown Curve   The convex longitudinal profile of the water surface in an open channel upstream from a sudden fall. (MTC)

Drop Structure   A structure in a culvert or channel at which water drops to a lower level. An inclined drop may also be termed a chute or fall. (MTC)

Ecosystem   A community of plants and animals interacting together with the physical and chemical environment. (FHWA)

Embankment   A bank of earth, rock or other material constructed above the natural ground surface. (MTC)

Emergency Spillway   A spillway to carry flood discharges exceeding a given value. (MTC)

Encroachment   1. The distance by which the flow in a gutter encroaches onto the traffic lanes. (MTC) 2. A highway action within the limits of a base flood plain. (AA)

End Section   A prefabricated end structure for a pipe. (MTC)

End Treatment   The overall design of the culvert ends and of erosion control measures for the adjoining fills. (MTC)

Energy Dissipator   A structure used to dissipate the energy possessed by high-velocity flow. (MTC)

Energy Grade Line   A hypothetical line representing the total energy at any point along a culvert barrel (or sewer or channel). (MTC)

Entrance Loss   The head lost in eddies and friction at the inlet to a conduit or structure. (AA)

EPA   1. Acronym for Ontario *Environmental Protection Act*. 2. Acronym for US Environmental Protection Agency.

Ephemeral Stream   A stream that flows only in direct response to precipitation, and whose channel is at all times above the water table. (AA)

Erosion   Displacement of soil particles on the land surface due to such things as water or wind action. (AA)

FHWA   Acronym for US Federal Highway Administration.

Filtration Opening Size   The diameter, in micrometre, of particles equal to a specified size (currently D_{95}) of the material passing the geotextile during a hydrodynamic sieving operation. (MTC)

Fish Passage Design Flow A relatively small flood or other flow representing flow conditions at the time of fish migration on a stream, and used for the purpose of checking that fish will be able to pass through a culvert during the migration period. (MTC)

Fishway A facility, such as a series of weirs and pools, to permit fish to pass an obstruction with minimum stress. (MTC)

Flood Frequency The number of times a flood event occurs or is exceeded during a given period. Frequency is the reciprocal of return period, but is often used synonymously with it. (MTC)

Flood Plain The area, usually low lands, adjoining a watercourse which has been, or may be, covered by flood water. (MNR)

Flood Rise The height of a flood measured from normal water level, or from the average bed if the stream is usually dry in the summer. (MTC)

Flood Routing The process of determining progressively the timing and shape of a flood wave at successive points along a river, a valley or a reservoir. (AA)

Floodway The channel of a watercourse and that portion of the flood plain where flood depths and velocities are generally higher than those experienced in the flood fringe. The floodway represents that area required for safe passage of flood flow and/or that area where flood depths and/or velocities are considered to be such that they pose a potential threat to life and/or property damage. (MNR)

Flume An open conduit of wood, concrete, metal, etc. on a prepared grade, trestle, or bridge. A flume holds water as a complete structure. (AA)

Fluvial Geomorphology A study of the structure and formation of the earth's features which result from the forces of water. (AA)

Frazil Ice A group of ice crystals having the form of small discs or spicules which is formed in supercooled turbulent water. These groups may accumulate on a water surface to form frazil slush. (MTC)

Free Flow A condition of flow through or over a structure not affected by tailwater. (MTC)

Freeboard The height from high water level to the top of an embankment, roadway, dam or wall. (MTC)

Frequency See Flood Frequency.

Friction Head Also called friction loss. The head or energy lost as the result of the disturbances set up by the contact between a moving stream of water and its container conduit or channel. In laminar flow, the friction head is approximately proportional to the first power of the velocity; in turbulent flow, to a higher power - practically the square. Friction losses are separate from losses due to bends, expansions, obstructions impacts, etc. (MTC)

Frost Heave A seasonal upthrust of the ground or pavement caused by the formation of ice layers or lenses in a frost susceptible soil. (MTC)

Froude Number A dimensionless ratio of inertia forces to gravity forces of flow, used as an index to characterize the type of flow in a stream or hydraulic structure. (ASCE)

General Scour   Scour which occurs in a waterway opening as a result of constriction of the flow. (MTC)

Geotextile   Fabric of synthetic material that serves the same purpose as a granular filter blanket. (AA)

Gradually Varied Flow   A flow condition in which the depth or velocity changes slowly over a long distance. (MTC)

Grate Inlet   Storm sewer inlet located in a gutter and covered by a grate. (MTC)

Groundwater   Subsurface water occupying the saturation zone, from which wells and springs are fed. A source of base flow in stream. (AA)

Groundwater Table   See Water Table.

Groyne   A structure built out from a bank or shoreline to control erosion and arresting sand movement along the shoreline. Also spelled as Groin. (MTC)

Guide Bank   A short embankment extending upstream from the river end of a bridge approach embankment approximately parallel to the stream bank. Also called Spur Dike. (MTC)

Gutter   The portion of roadway adjoining the curb for conveying surface runoff. (MTC)

Gutter Inlet   A storm sewer inlet in or immediately adjacent to a gutter. (MTC)

Hazel Flood   A flood generated by the Hurricane Hazel storm (1954) centred on the watershed in question, used for flood regulatory purposes. (MTC)

Head Cutting   Progressive degradation of a stream bed at a relatively rapid rate in an upstream direction, usually characterized by one or more vertical drops ranging from a few centimetres to a few metres. (MTC)

Headwall   A wall at the end of a culvert normally extending from the invert to above the soffit or crown of the culvert, and aligned parallel to the roadway or normal to the longitudinal axis of the culvert. (MTC)

Headwater   The water upstream from a culvert or other structure. (MTC)

High Water Mark   Evidence such as debris, ice scars indicating a high water level. (MTC)

High Water Level   The highest level reached by a flood. (MTC)

Hydraulic Depth   The ratio of the cross sectional water area in a stream or channel to the width of the water surface. (MTC)

Hydraulic Grade Line   A hypothetical line indicating the levels to which water would rise in a series of small vertical pipes attached to a culvert along its length. The water surface profile of an open channel.

(MTC)

Hydraulic Jump An abrupt rise in water surface that occurs when flow changes from supercritical to subcritical. (MTC)

Hydraulic Radius The ratio of the water cross sectional area of a stream or conduit divided by its wetted perimeter. (AA)

Hydraulics The science dealing primarily with the flow of liquids. (MTC)

Hydrograph A graph showing stage, flow, velocity or other characteristics of water with respect to time. (AA)

Hydrologic Soil Group A classification of a soil type on the basis of its permeability after prior wetting and swelling of the soil, and without the protective effect of vegetation. (MTC)

Hydrology The science dealing with the waters of the earth, their occurrence, circulation and distribution, their chemical and physical properties, and their reaction with the environment, including their relation to living things. The domain of hydrology embraces the full life history of water on the earth. (FCST)

Hydroplaning The phenomenon in which a moving vehicle tire is supported by a film of water between it and the pavement. (MTC)

Hyetograph A graphical representation of average rainfall or rainfall excess rates over successive increments of a storm. (MTC)

Ice Jam The choking of a stream channel by the piling up of ice at an obstruction or constriction. (MTC)

Improved Inlet A culvert inlet incorporating geometric refinements other than those used in conventional culvert practice, for the purpose of improving the culvert's capacity. (MTC)

Incised Channel A deep and relatively straight stream channel with high and steep banks such that overbank flow rarely occurs. (AA)

Infiltration Capacity The maximum rate at which a soil is capable of absorbing water. (AA)

Infiltration Rate The rate at which water enters the soil under a given condition, e.g. prevailing soil moisture content. (AA)

Inlet Capacity The rate of flow entering an inlet under a given set of gutter flow conditions. (MTC)

Inlet Control A condition where the relation between the headwater elevation and discharge is controlled by the upstream end of any structure through which water may flow. (AA)

Inlet Time The time for stormwater to flow from the most distant point in a drainage area to the point at which it enters a storm drain. (AA)

Intensity-Duration-Frequency Curve   Curves (or tables) expressing rainfall intensities for specified durations and frequencies. (MTC)

Invert   The stream bed or floor within a structure or channel. (MTC)

Jackson Turbidity Unit (JTU)   See Turbidity.

K Factor   See Soil Erodibility Factor.

Lag Time   Variously defined as the time from the beginning of rainfall to the peak (or centre of mass) of runoff. (AA)

Launching Apron   A horizontal layer of flexible material designed to settle and protect the sides of a scour hole as scour progresses. (MTC)

Live Cribwall   A rectangular framework of logs or timbers constructed with living woody plant cuttings that are capable of rooting. (MTC)

Live Fascine   Sausage-like bundles of living woody plant cutting tied together. These fabricated structures are capable of rooting. (MTC)

Live Stake   Living woody plant cutting taken from living shrubs and trees, that are capable of rooting. (MTC)

Local Scour   Scour which occurs at a pier or abutment as a result of local obstruction to the flow, and which is measured below the level of general scour adjacent to the pier or abutment. (MTC)

Low Flow Channel   A small channel in the bottom of a larger channel for conveying dry-weather discharges. (MTC)

LS Factor   A factor representing the combined effect of the steepness and length of a slope (measured down the slope) on the rate of erosion. Also called topographic factor. (MTC)

Maintenance Hole   An opening through which a person may gain access to a sewer for inspection and maintenance purposes. A removable cover is provided. (MTC)

Major System   The route followed by runoff when the capacity of the minor system is exceeded, and generally comprising the roads and major drainage channels. (MTC)

Manning Equation   An empirical formula devised by Manning for calculating flow in open channels and conduits. (MTC)

Manning's n   The roughness coefficient for use with Manning equation. (MTC)

Mean Annual Discharge   A flood equal to the mean of the discharges of all the maximum annual floods during the period of record. (ASCE)

Meander   A loop-like bend in a sinuous stream channel. (MTC)

Meander Length   Distance along a stream between corresponding points of successive meanders of the same amplitude. (AA)

Median   The portion of a divided highway separating roadways carrying traffic in opposite directions. (MTC)

Minor System   The drainage system provided to accommodate relatively minor floods (2- to 10-year return period), and comprising the gutters, catchbasins, storm sewers, and minor channels. (MTC)

Model   A conceptual description of a surface water system and the associated mathematical representation of the response of the system to the physical processes affecting the system. An example is the computation of a hydrograph resulting from a design storm. (AA)

Natural Drainage Concept   The concept of incorporating attributes of natural drainage into the design of new drainage systems. (MTC)

Natural Scour   Scour of a stream bed resulting from natural phenomena such as channel meandering. (MTC)

Nephelometric Turbidity Unit (NTU)   See Turbidity.

Normal Depth   The depth of flow corresponding to a given discharge in an open channel under uniform flow conditions. (MTC)

Normal Flow   1. The flow that prevails the greatest portion of the time; the mean flow. 2. Flow at normal depth. (ASCE)

NURP   Acronym for "Nationwide Urban Runoff Program" sponsored by US EPA in the early 1980's.

One Dimensional Flow   An idealized flow in which the velocity is assumed to be constant across the channel cross section and equal to the mean velocity. (MTC)

Open Channel Flow   Flow having its surface exposed to atmospheric pressure; the flow may be in an open channel or in a pipe flowing partially full. (MTC)

Open Footing Culvert   A culvert having either a natural invert or an artificial floor not integrated with the walls. (MTC)

Orifice Flow   Flow similar to that through an orifice. The inlet is completely submerged but the conduit downstream from the orifice is only partially full. (ASCE)

Outlet Control   A condition where the relation between headwater and discharge is controlled by the conduit, outlet, or downstream conditions of any structure through which water may flow. (AA)

Overland Flow   The flow of water over the ground before it enters into a conduit. (ASCE)

Peaking Factor   The ratio of instantaneous (peak) discharge to daily (average) discharge. (MTC)

Pervious   Applied to materials (e.g. soil) through which water passes relatively freely. (MTC)

Point Bar   A crescent-shaped deposit of sediment accumulated on the inside of a meander loop. (MTC)

Pool   A small, rather deep body of quiescent water, as a pool in a stream. See also Riffle. (AA)

Prescriptive Right   Title or right acquired through its open and continued use or actual possession from time immemorial or over a legally recognized or prescribed period. (MTC)

Pressure Conduit Flow   Surcharged flow in a pipe, i.e. flow under hydrostatic pressure. (MTC)

Pressure Head   Hydrostatic pressure expressed as the height of a column of water that pressure can support at the point of measurement. The height at any point in a conduit represented by the height of the hydraulic grade line above that point. (AA)

Probability Distribution   A mathematical function describing the relative frequency with which events of various magnitudes occur. Probability distributions of interest to hydraulics and hydrology are: normal distributions, Pearson distributions, extremal distributions, and logarithmic distributions. (AA)
Probability of Occurrence   The probability that a flood of a given magnitude will be equalled or exceeded in a given period. The probability is the reciprocal of the return period. (MTC)

Rapidly Varied Flow   A flow condition where the depth or velocity changes abruptly over a relatively short distance. (MTC)

Rating Curve   A graph (or table) of the discharge of a river at a particular point as a function of the elevation of the water surface. (AA)

Reach   A length of a stream selected for use in hydraulic computations or other purposes. (MTC)

Regime   The condition of a stream with regard to its stability. A stream is "in regime" if its channel has reached a stable form as a result of its flow characteristics. (AA)

Regulatory Flood   The approved standard(s) used in a particular watershed to define the limit of the flood plain for regulatory purposes. (MNR)

Relief Flow   The flood flow which bypasses the main structure at a stream crossing by flowing over the roadway or through a relief bridge or culvert. (MTC)

Return Period   See Flood Frequency and Probability of Occurrence.

Revetment   A vertical or inclined facing of riprap or other material protecting a soil surface from erosion. (MTC)

Riffle   A rapid in a stream. Shallow rapids in an open channel, where the water surface is broken into waves by obstructions wholly or partly submerged. Typically, riffles alternate with pools along the length of a channel. (AA)

Riparian Right A right of the owners of lands along watercourses, relating to water, its use, ownership of soil under the stream, accretion, etc. (MTC)

Riprap A layer of stone or broken concrete to prevent the erosion of soil. (MTC)

Runoff Coefficient A coefficient in the Rational formula expressing the ratio of the depth of runoff from a drainage basin to the depth of rainfall, and indicating the runoff potential of particular topography/soil type/ land use combination. (MTC)

Runoff See Surface Runoff.

Sag The low point in a road profile to which runoff flows both directions. (MTC)

Saltation Bed load particles that skip along the stream bed while transported by the flow. (AA)

Scour Local lowering of a stream bed by the erosive action of flowing water. (MTC)

Scour Depth The depth of material removed from a stream bed by scour, measured from the original bed elevation. (MTC)

Scoured Depth The total depth of water measured from high water level to scoured bed. (MTC)

Sediment Mineral or organic matter that is transported or deposited by, or suspended in, water or other agents. (MTC)

Sediment Delivery Ratio The ratio of the amount of sediment delivered to a given point to the soil loss from a given area. (MTC)

Sediment Load See Total Sediment Discharge.

Sediment Transport Soil particles being transported away from their natural location by wind or water action. (MTC)

Sediment Yield The soil losses from an area minus deposition in ditches, buffer strips, sediment traps, barriers, basins and channels. (MTC)

Sedimentation The deposition of detached soil particles. (MTC)

Sequent Depth See Conjugate Depth.

Shear Stress The tractive force or drag (per unit area) on a stream bank caused by passing water which tends to pull soil particles along with the stream flow. The force or drag developed at the channel bed by flowing water. (AA)

Sheet Flow Water, usually in the form of storm runoff, flowing in a thin uniform sheet over a ground surface. (MTC)

Silt Curtain   A curtain of fabric supported by posts or a cable and anchor system in static or very slow moving water to confine suspended sediment within a given area. (MTC)

Simulation   See Model.

Sinuosity   The ratio of the length of the stream thalweg to the length of the valley proper. (MTC)

Skew   The measure of the angle of intersection between a line normal to the roadway centreline and the direction of the flow in a channel at flood stage in the lineal direction of the main channel. (AA)

Slope Drain   A channel or pipe for transporting water down or draining a cut or fill slope. (MTC)

Sloughing   Slow crumbling and falling away of the surface of an earth bank, usually as a result of weakening by groundwater. (MTC)

Slump   The downward slipping of part of a slope along a plane that is curved concavely upward, caused by a lack of underlying support and/or seepage of groundwater. The movement may occur over a long period or in a single rapid event. (MTC)

Soffit   The undersurface of the superstructure of a bridge or culvert. (MTC)

Soil Bioengineering   An applied science that uses living plant material as a main structural component. (MTC)

Soil Erodibility Factor K   A factor representing the erodibility of a soil arising from the properties of the soil rather than from factors such as slope or surface treatment. (MTC)

Sounding   A measurement of the depth of water or soft stream bed material by a wire and weight, rod, echo sounder or other means. (MTC)

Southern Type Basin   A watershed not having the characteristics of a Canadian Shield or Hudson Bay Lowland type basin. (MTC)

Spillthrough Abutment   A bridge abutment having a fill slope on the stream side. The term originally referred to the "spillthrough" of fill at an open abutment but is now used for any abutment having such a slope. (AA)

Spillway   A relatively steep open or closed conduit to convey water from a higher to a lower level. (MTC)

Spread   The accumulated flow in and next to the roadway gutter. The transverse encroachment of stormwater onto a street. The lateral distance, in metres, of roadway ponding extending out from the curb or edge of the travelled way. (AA)

Spread Footing   A pier or abutment footing that transfers load directly to the earth. (AA)

Spur Dike See Guide Bank.

SS See Suspended Solids.

Stage The height of a water surface above a specified datum. (MTC)

Standard Deviation A measure of the dispersion of a series of statistical values such as precipitation, stream flow, etc. (ASCE)

Steady Flow Flow in which the discharge at a given point remains constant with time. (MTC)

Stilling Basin An open basin or structure at the outlet of a chute, drop, spillway or other facilities to reduce the energy of the flow. (MTC)

Stream, Ephemeral See Ephemeral Stream

Stream Morphology See Fluvial Geomorphology

Subcritical Flow In this state, gravity forces are dominant so that the flow has a relatively low velocity and is often described as tranquil or streaming. Also, that flow which has a Froude number less than unity. Flow at velocities less than critical velocity; flow at depths greater than critical depth. (AA)

Subgrade The soil or rock supporting the pavement structure. Sometimes used to denote the upper surface of the supporting material. (MTC)

Subsurface Drainage The removal of excess water from below a pavement or soil surface. (MTC)

Supercritical Flow In this state, inertia forces are dominant so that flow has a high velocity and is usually described as rapid or shooting. Also, that flow which has a Froude number greater than unity. Flow at velocities greater than critical velocity; flow at depths less than critical depth. (AA)

Superelevation 1. Local increase in water surface on the outside of a stream or channel bend. 2. Increase in elevation at the outside edge of a road at a horizontal curve. (AA)

Surcharged Flow Flow in a closed conduit which is under pressure greater than atmospheric. (MTC)

Surface Runoff That portion of rainfall that moves over the ground toward a lower elevation and does not infiltrate the soil. (MTC)

Suspended Solids The quantity of material removed from wastewater or stormwater runoff in a laboratory test under preset test conditions. (FHWA)

Swamp A form of wetland typically vegetated by trees which are rooted in mineral soil. (MTC)

Synthetic Design Storm See Design Storm.

Tailwater The water immediately downstream from a culvert or other structure. (MTC)

Tailwater Depth The depth of tailwater measured from the invert of the culvert or other structure. (MTC)

Thalweg A line joining the lowest points along a channel or valley. (MTC)

Throat Control A type of inlet control in an improved inlet culvert in which the capacity is governed by the throat characteristics and the headwater depth. (MTC)

Throat Section The intersection of the sidewall tapers and culvert barrel in a side- or slope-tapered inlet. (MTC)

Time of Concentration The time taken for the storm runoff to travel from the farthest point of the basin to the site in question, the farthest point being determined on the basis of travel time and not necessarily distance. (MTC)

Timmins Flood A flood generated by the Timmins storm (1961) centred on the watershed in question and used for regulatory purposes. (MTC)

Topsoil The upper layer of soil containing organic matter and suited for plant growth. (MTC)

Tort A private or civil wrong committed upon the person or property independent of contract. The elements of every tort action are: existence of legal duty from defendant to plaintiff, breach of duty, and damage as proximate result. (MTC)

Total Sediment Discharge The sum of the suspended sediment discharge and the bed load discharge. Loosely termed (total) sediment load. (MTC)

Tractive Force See Shear Stress.

Transition A short conduit or channel uniting two others having different hydraulic elements; a connection. (MTC)

Turbidity A measure of the light-scattering ability of material suspended in water. Two commonly known units of measurement are the Jackson Turbidity Unit (JTU) and the Nephelometric Turbidity Unit (NTU). NTU is the current practice. (F, AA)

Uniform Flow Flow in which the velocities are uniform in both magnitude and direction along a conduit, all stream lines being parallel. (MTC)

Unit Hydrograph The hydrograph of direct runoff from a storm uniformly distributed over the drainage basin during a specified unit of time; the hydrograph is reduced in vertical scale to correspond to a volume of runoff of one mm from the drainage basin. (AA)

Universal Soil Loss Equation An equation designed to predict the long-term average soil loss caused by overland flow on a soil surface. (MTC)

Unsteady Flow Flow in which the velocity at a given point varies with time. (MTC)

Uplift See Culvert Uplift.

Velocity Head   The kinetic energy of flowing water expressed in metres. (MTC)

Velocity, Mean   The velocity of flow obtained by dividing the flow rate by the flow area. (MTC)

VM Factor   A factor characterizing the effects of erosion control measures such as vegetative and non-vegetative soil covers and mechanic manipulation of the soil surface. Also termed cover factor or erosion control factor. (MTC)

Warm Water Fishery   A fresh water, mixed fish population with no salmonoids. (MOEa)

Wash Load   That portion of the total sediment load that is composed of particle sizes finer than those of the stream bed. (AA)

Washout   The failure of a culvert, bridge, embankment or other structure resulting from the action of flowing water. (MTC)

Water Table   The upper surface of the zone of saturation of the soil where the groundwater is not confined by an overlying impermeable formation. If an overlying confining formation is present, the aquifer in question has no water table. (MTC)

Watercourse   A stream, river or channel in which a flow of water occurs, either continuously or intermittently, with some degree of regularity. (AA)

Watershed   All land drained by a river or stream and its tributaries. (MNR)

Watershed Class   Classification of a watershed based on its physical characteristics. (MTC)

Watershed Length   The length of the main channel measured from the head of the watershed to the site under consideration. (MTC)

Watershed Slope   The representative slope of the main channel from the head of the watershed to the point under consideration. (MTC)

Waterway Area (Scoured)   The net scoured cross sectional area of a bridge waterway normal to the flow and measured from the high water level (or soffit if lower) to the scoured bed. (MTC)

Waterway Area (Unscoured)   The net unscoured cross sectional area of a bridge waterway normal to the flow and measured from the high water level (or soffit if lower) to the original (unscoured) stream bed.

(MTC)

Waterway, Bridge   See Bridge Waterway.

Weir   A dam or device installed across a stream or channel for diverting or measuring the flow. (MTC)

Wetland   An area that is inundated or saturated by surface waters or groundwater at a frequency and duration sufficient to support a prevalence of vegetation typically adapted for life in saturated soil conditions. Wetlands generally include swamps, marshes, bogs, and similar areas. Wetlands typically have hydric soils, phreatic vegetation, and wetland hydrology. (AA)

Wetted Perimeter The length of the wetted contact between a stream of water and its containing conduit, measured along a plain at rights to the flow in question. (AA)

Wind Setdown A temporary lowering of the static level of a lake at its upwind end caused by the transfer of surface water to the downwind end by the action of wind. (MTC)

Wind Setup The raising of the water level on the downwind end of a lake due to the action of wind on the water surface. (MTC)

Acknowledgement

This glossary has been compiled from the following sources:

AA	AASHTO, (1992). *Drainage Guidelines*, Glossary.
ASCE	American Society of Civil Engineers, (1962). *Nomenclature for Hydraulics*.
FCST	US Federal Council for Science and Technology (1962). *Scientific Hydrology*, Washington D.C.
FHWA	US Federal Highway Administration, (undated). *Highway Runoff Water Quality Training Course, Student Workbook*, National Highway Institute, Washington, D.C.
MNR	Ontario Ministry of Natural Resources, (1988). *Flood Plain Planning Policy Statement*.
MOEa	Ontario Ministry of the Environment and Energy, (1984). *Water management: Goals, Policies, Objectives and Implementation Procedures*.
MOEb	Ontario Ministry of the Environment and Energy, (1991). *Stormwater Quality Best Management Practices.*
MTC	Ontario Ministry of Transportation, (1980-88, 1992). *MTC Drainage Manual*

Combined Index

Notation:
The number *m.n* means page n of Chapter m.

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