SMALL DAM DESIGN AND CONSTRUCTION MANUAL

MAY, 1992
Agriculture Canada
Prairie Farm Rehabilitation Administration
Prairie Resources Service

SMALL DAM DESIGN AND CONSTRUCTION MANUAL

May, 1992
<table>
<thead>
<tr>
<th>Revision Number</th>
<th>Description</th>
<th>Date</th>
<th>By</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Addition of Appendix “F” - Updated Hydrology Section</td>
<td>97/10/06</td>
<td>B. Bell</td>
</tr>
<tr>
<td>2.</td>
<td>Appendix “G” - Federal Environment Assessment Process</td>
<td>97/10/06</td>
<td>E. Kienholz</td>
</tr>
</tbody>
</table>
This manual has been developed by PFRA specifically for use by its staff in the design and construction of the small dams built under various PFRA water development and conservation programs. The investigation and design phases and the technical assistance during the construction of these dams, as described in the manual, are organized based on precedent delivery modes of these programs. The manual relies on the technical skills presently in place in the PFRA district offices and the engineering expertise available in headquarters and the regional offices.

Any revisions to the recommended organization of the investigation, design and construction roles, made necessary by changes in PFRA programs or redistribution of staff, will be incorporated into the manual through amendments.

The manual presents engineering information for the design, construction, operation and maintenance of small dams on the Canadian Prairies. Under certain limiting conditions that consider size, potential hazards, construction materials and level of investment, the manual provides the basis for safe, cost-effective projects which can be designed by qualified engineering technicians or technologists, subject to the review of a qualified professional engineer. The manual is arranged in logical, convenient sections, dealing with investigations, design, construction, operation and maintenance, and includes various appendices presenting design aids and additional information.

The text was prepared by an engineering team comprising PFRA engineers under the overall direction of D. H. Pollock, Chief, Technical Resources, Development Service. The text was coordinated and edited by C. R. Campbell. Editorial guidance and review was provided by R. R. Weinberger. Other reviewers included J. Lebedin, F. R. J. Martin, M. Lu and R. E. Lien. The authors and their areas of contribution are:

M. Mowchenko - hydrology;
J. C. Oosterveen and T. A. Dash - geology;
V. Klassen - geotechnical design, construction; and
C. Campbell - hydraulic and structural design, construction, operation and maintenance.

As PFRA cannot satisfy itself that the manual will not at some time be acquired and used inappropriately by others, PFRA shall be under no liability whatsoever to any individual or group using this manual, and makes no warranty concerning the completeness, reliability or suitability of the information contained herein.
# TABLE OF CONTENTS

## 1.0 INTRODUCTION

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1 General</td>
<td>1-1</td>
</tr>
<tr>
<td>1.2 Application of the Manual</td>
<td>1-3</td>
</tr>
<tr>
<td>1.2.1 Project Concept</td>
<td>1-3</td>
</tr>
<tr>
<td>1.2.2 Foundations and Embankment Materials</td>
<td>1-4</td>
</tr>
<tr>
<td>1.2.3 Size Classification</td>
<td>1-4</td>
</tr>
<tr>
<td>1.2.4 Hazard Potential Classification</td>
<td>1-4</td>
</tr>
<tr>
<td>1.3 Design Responsibility</td>
<td>1-5</td>
</tr>
<tr>
<td>1.4 Project Life Cycle</td>
<td>1-6</td>
</tr>
</tbody>
</table>

## 2.0 INVESTIGATIONS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1 General</td>
<td>2-1</td>
</tr>
<tr>
<td>2.2 Hazard Potential</td>
<td>2-2</td>
</tr>
<tr>
<td>2.2.1 Methods For Determining Hazard Potential</td>
<td>2-2</td>
</tr>
<tr>
<td>2.2.2 Hazard Potential Assessment</td>
<td>2-3</td>
</tr>
<tr>
<td>2.3 Geology</td>
<td>2-7</td>
</tr>
<tr>
<td>2.3.1 Classification and Engineering Characteristics of Geological Materials</td>
<td>2-7</td>
</tr>
<tr>
<td>2.3.2 Common Relationships Among Geologic Unit</td>
<td>2-16</td>
</tr>
<tr>
<td>2.3.3 Errors in Interpretation of Relationships</td>
<td>2-22</td>
</tr>
<tr>
<td>2.3.4 Geological Damsite Description</td>
<td>2-22</td>
</tr>
<tr>
<td>2.4 Hydrology</td>
<td>2-29</td>
</tr>
<tr>
<td>2.4.1 Flood Potential</td>
<td>2-30</td>
</tr>
<tr>
<td>2.4.1.1 General Concepts</td>
<td>2-30</td>
</tr>
<tr>
<td>2.4.1.2 Development of Flood Determination Model</td>
<td>2-31</td>
</tr>
<tr>
<td>2.4.1.3 Procedures for Flood Peak Determination</td>
<td>2-33</td>
</tr>
<tr>
<td>2.4.1.4 Example of Flood Peak Calculation</td>
<td>2-35</td>
</tr>
<tr>
<td>2.4.1.5 Uses and Limitations</td>
<td>2-36</td>
</tr>
<tr>
<td>2.4.2 Water Supply Potential</td>
<td>2-36</td>
</tr>
<tr>
<td>2.4.2.1 General Concepts</td>
<td>2-36</td>
</tr>
<tr>
<td>2.4.2.2 Development of Water Supply Determination Procedures</td>
<td>2-38</td>
</tr>
</tbody>
</table>
# TABLE OF CONTENTS (Cont'd)

<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.4.2.3</td>
<td>Procedures for Water Supply</td>
<td>2-40</td>
</tr>
<tr>
<td>2.4.2.4</td>
<td>Examples of Water Supply</td>
<td>2-41</td>
</tr>
<tr>
<td>2.4.2.5</td>
<td>Users and Limitations</td>
<td>2-44</td>
</tr>
<tr>
<td>2.4.3</td>
<td>Hydrology Summary Report</td>
<td>2-45</td>
</tr>
<tr>
<td>2.5</td>
<td>Geotechnology</td>
<td>2-63</td>
</tr>
<tr>
<td>2.5.1</td>
<td>Soil Classification</td>
<td>2-63</td>
</tr>
<tr>
<td>2.5.1.1</td>
<td>Laboratory Tests</td>
<td>2-63</td>
</tr>
<tr>
<td>2.5.1.2</td>
<td>Identification of Soil Groups</td>
<td>2-64</td>
</tr>
<tr>
<td>2.5.2</td>
<td>Field Investigations</td>
<td>2-70</td>
</tr>
<tr>
<td>2.5.2.1</td>
<td>Reconnaissance Investigations</td>
<td>2-70</td>
</tr>
<tr>
<td>2.5.2.2</td>
<td>Subsurface Investigations</td>
<td>2-73</td>
</tr>
<tr>
<td>2.5.3</td>
<td>Types of Foundations</td>
<td>2-76</td>
</tr>
<tr>
<td>2.5.4</td>
<td>Geotechnical Assessment</td>
<td>2-77</td>
</tr>
<tr>
<td>2.6</td>
<td>Surveys and Mapping</td>
<td>2-89</td>
</tr>
<tr>
<td>2.7</td>
<td>Site Investigation Guidelines</td>
<td>2-91</td>
</tr>
</tbody>
</table>

## 3.0 DESIGN

<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.1</td>
<td>General</td>
<td>3-1</td>
</tr>
<tr>
<td>3.2</td>
<td>Design Inputs</td>
<td>3-3</td>
</tr>
<tr>
<td>3.3</td>
<td>Embankment Design</td>
<td>3-4</td>
</tr>
<tr>
<td>3.3.1</td>
<td>Description</td>
<td>3-4</td>
</tr>
<tr>
<td>3.3.2</td>
<td>Criteria and Standards</td>
<td>3-4</td>
</tr>
<tr>
<td>3.3.3</td>
<td>Methods</td>
<td>3-9</td>
</tr>
<tr>
<td>3.4</td>
<td>Spillway System Design</td>
<td>3-15</td>
</tr>
<tr>
<td>3.4.1</td>
<td>Description</td>
<td>3-15</td>
</tr>
<tr>
<td>3.4.2</td>
<td>Criteria</td>
<td>3-16</td>
</tr>
<tr>
<td>3.4.3</td>
<td>Standard Spillway Structures</td>
<td>3-21</td>
</tr>
<tr>
<td>3.4.4</td>
<td>Methods</td>
<td>3-21</td>
</tr>
<tr>
<td>3.5</td>
<td>Reservoir Outlet Design</td>
<td>3-27</td>
</tr>
<tr>
<td>3.5.1</td>
<td>Description</td>
<td>3-27</td>
</tr>
<tr>
<td>3.5.2</td>
<td>Criteria</td>
<td>3-27</td>
</tr>
<tr>
<td>3.5.3</td>
<td>Standard Reservoir Outlet Structures</td>
<td>3-28</td>
</tr>
</tbody>
</table>
### TABLE OF CONTENTS (Cont'd)

<table>
<thead>
<tr>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.5.4 Methods ........................................ 3-29</td>
</tr>
<tr>
<td>3.6 Design Output ...................................... 3-30</td>
</tr>
</tbody>
</table>

#### 4.0 CONSTRUCTION

<table>
<thead>
<tr>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.1 General ............................................ 4-1</td>
</tr>
<tr>
<td>4.2 Earthwork .......................................... 4-2</td>
</tr>
<tr>
<td>4.3 Quality Control Inspection for Earthwork Construction ...................................... 4-7</td>
</tr>
<tr>
<td>4.4 Concrete ............................................ 4-16</td>
</tr>
<tr>
<td>4.5 Construction Supervision and Quality Control Testing ...................................... 4-17</td>
</tr>
<tr>
<td>4.6 As-Constructed Drawings ...................................... 4-18</td>
</tr>
</tbody>
</table>

#### 5.0 OPERATION AND MAINTENANCE

<table>
<thead>
<tr>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.1 General ............................................ 5-1</td>
</tr>
<tr>
<td>5.2 Operation .......................................... 5-2</td>
</tr>
<tr>
<td>5.3 Maintenance ........................................ 5-3</td>
</tr>
<tr>
<td>5.4 Inspection ......................................... 5-4</td>
</tr>
<tr>
<td>5.5 Operation and Maintenance Manual Model ...................................... 5-5</td>
</tr>
</tbody>
</table>

Appendix A Standard Drawings
Appendix B Specifications for Construction
Appendix C Cost Estimating
Appendix D Design Example
Appendix E Definitions
References
Attachments
# LIST OF TABLES

<table>
<thead>
<tr>
<th>Table No.</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.2.1</td>
<td>Project Hazard Potential Classification</td>
</tr>
<tr>
<td>2.4.1</td>
<td>Drainability Factors Selection Chart</td>
</tr>
<tr>
<td>2.4.2</td>
<td>Elevation-Storage-Area Relationship for Standard Reservoir</td>
</tr>
<tr>
<td>2.4.3</td>
<td>Table of Divertible Flow Factors</td>
</tr>
<tr>
<td>2.5.1</td>
<td>Soil Classification and Description Chart</td>
</tr>
<tr>
<td>2.5.2</td>
<td>Auxiliary Laboratory Identification Procedure</td>
</tr>
<tr>
<td>2.5.3</td>
<td>Classification of Foundations and Embankment Materials for Homogeneous Small Storage Dams</td>
</tr>
<tr>
<td>3.3.1</td>
<td>Design Standards for Upstream Slope Protection</td>
</tr>
<tr>
<td>3.4.1</td>
<td>General Guide to Spillway Selection</td>
</tr>
<tr>
<td>3.4.2</td>
<td>Design Floods</td>
</tr>
</tbody>
</table>

D.1 Preliminary Cost Estimate - Joe Smith Dam
### LIST OF FIGURES

<table>
<thead>
<tr>
<th>Figure No.</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.4.1</td>
<td>Project Life Cycle</td>
</tr>
<tr>
<td>1.4.2</td>
<td>Project Flow Chart</td>
</tr>
<tr>
<td>2.2.1</td>
<td>Hazard Potential Assessment Summary</td>
</tr>
<tr>
<td>2.3.1</td>
<td>Two Major Groups of Geologic Units</td>
</tr>
<tr>
<td>2.3.2</td>
<td>Disturbance Due to Mountain-Building Forces</td>
</tr>
<tr>
<td>2.3.3</td>
<td>Disturbance Due to Salt-Solution Collapse</td>
</tr>
<tr>
<td>2.3.4</td>
<td>Disturbance Due to Glacial Thrusting</td>
</tr>
<tr>
<td>2.3.5</td>
<td>Minor Pre-Valley Units and Their Relationship to Major Pre-Valley Units</td>
</tr>
<tr>
<td>2.3.6</td>
<td>Valley Units</td>
</tr>
<tr>
<td>2.3.7</td>
<td>Channel (Sand &amp; Gravel) and Floodplain (Clay &amp; Silt) Units</td>
</tr>
<tr>
<td>2.3.8</td>
<td>Migration of Dune Sand Down Flank of Valley and onto Floodplain</td>
</tr>
<tr>
<td>2.3.9</td>
<td>Various Relationships for Slump Debris</td>
</tr>
<tr>
<td>2.3.10</td>
<td>Some Hypothetical Opportunities for Error in Interpretation</td>
</tr>
<tr>
<td>2.3.11</td>
<td>Geological Damsite Description - Summary Sheet</td>
</tr>
<tr>
<td>2.4.1</td>
<td>Index Flood Selection Graph</td>
</tr>
<tr>
<td>2.4.2</td>
<td>Index Flood Multiplier Selection Graph</td>
</tr>
<tr>
<td>2.4.3</td>
<td>Rainfall Isopleths for a 1:100 Six-hour Rain</td>
</tr>
<tr>
<td>2.4.4</td>
<td>Flood Peak Adjustment Factor Based on the Rainfall Component</td>
</tr>
<tr>
<td>2.4.5</td>
<td>Isopleths of Average May-June Net Evaporation</td>
</tr>
<tr>
<td>2.4.6</td>
<td>Flood Peak Adjustment Factor Soil Moisture Component</td>
</tr>
<tr>
<td>2.4.7</td>
<td>Median Annual Unit Runoff for the Prairie Provinces</td>
</tr>
<tr>
<td>2.4.8</td>
<td>Runoff-Draft Regions for the Prairie Provinces</td>
</tr>
<tr>
<td>2.4.9</td>
<td>Storage-Draft Curves for Region No. 1</td>
</tr>
<tr>
<td>2.4.10</td>
<td>Storage-Draft Curves for Region No. 2</td>
</tr>
<tr>
<td>2.4.11</td>
<td>Storage-Draft Curves for Region No. 3</td>
</tr>
<tr>
<td>2.4.12</td>
<td>Storage-Draft Curves for Region No. 4</td>
</tr>
</tbody>
</table>
LIST OF FIGURES (Cont’d)

Figure No.
2.4.13 Storage-Draft Curves for Region No. 5
2.4.14 Hydrology Summary Report
2.5.1 Grain-Size Distribution Curve
2.5.2 Proposed Hole Locations for an Investigation of a Small Dam
2.5.3 Example PFRA Testhole Log
2.5.4 Geotechnical Assessment Report
3.3.1 Configurations for Internal Filter Systems
3.3.2 Proposed Embankment Design Cross-Section Example
3.4.1 Freeboard Allowances
3.4.2 Flow Chart For Spillway Design
4.2.1 Inclined Filter Construction Methods
4.3.1 Volumeter Field Density Test Apparatus
4.3.2 Standard Proctor Laboratory Test Apparatus
4.3.3 Typical Five-Point Standard Proctor Curve
4.3.4 One-Point Proctor Field Test for Clays
D.1 Project Area Sketch
D.2 Project Watershed
D.3 Stereogram
D.4 Hazard Potential Impacts
D.5 Hazard Potential Assessment Summary (Joe Smith Dam)
D.6 Geological Damsite Description - Summary Sheet (Joe Smith Dam)
D.7A PFRA Testhole Log (Joe Smith Dam)
D.7B PFRA Testhole Log (Joe Smith Dam)
D.8A Geotechnical Assessment Report (Joe Smith Dam)
D.8B Other Soil Logs
D.9 Capacity Curves
D.10 Weighted Average Back Slope
D.11 Hydrology Summary Report (Joe Smith Dam)
1.0 INTRODUCTION

1.1 GENERAL

Since inception in 1935, the Prairie Farm Rehabilitation Administration (PFRA) has been involved with dams - planning, designing, constructing, operating, inspecting and maintaining. More than 12,000 on-farm, group, and community dams have been built throughout the Canadian Prairies, all but a few with a successful record of operation and performance. Currently, all PFRA dams above a minimum size with the potential to impact on public safety are designed by a qualified, multi-discipline engineering team. On-farm dam facilities of lesser size which would not impact on the public safety are designed by technicians and technologists within various PFRA organizational areas and reviewed by a qualified professional engineer to ensure designs represent good engineering practice.

A technical reference for small dams was developed earlier by the PFRA Conservation Service and has been used successfully. However, in order to further utilize the PFRA experience accrued to date as well as latest developments in dam engineering technology, and to reflect the progress made in the area of "dam safety", the Development Service reviewed the overall design criteria, standards and procedures pertinent to small dam projects and prepared for PFRA use the "Small Dam Design and Construction Manual". The need to perform investigations, designs and inspections in the most effective and efficient manner, and to ensure consistency in certain aspects of these tasks have also led to the preparation of this manual.

The delivery modes of the Rural Water Development Program and other PFRA programs under which small dams are constructed have been taken into account in organizing the investigation and design phases and the inspection roles in the construction phase. The distribution and degree of technical skills in the PFRA district offices and availability of engineering expertise in headquarters and the regional offices were also taken into consideration in this aspect of the manual's preparation. Users of the manual should note, however, that the delivery modes of these programs and the organization of the agency may undergo change over time. While the policy is to update the manual whenever practical to reflect the consequences of any change, program managers are ultimately responsible for ensuring that no aspect of the manual is used improperly with respect to the degree of training and experience that is required for safe, cost-effective designs and sound construction practices.

The "Small Dam Design and Construction Manual" provides engineering criteria, methods, guidelines, and standards for the planning, design, construction, operation, and maintenance of relatively small storage dam facilities. The content and form of the manual enables experienced, suitably trained technicians and technologists to efficiently produce practical, cost-effective and safe designs, and to complete the detailed construction drawings and specifications required for construction.
The subject of project requirements and objectives, including assessment of agricultural and other benefits and any environmental aspects, is not considered in the manual. Although these items may impact on project details and viability, they are beyond the scope of this manual and would necessarily be considered at the initiation of the project and throughout the design process as required.
1.2 **APPLICATION OF THE MANUAL**

The "Small Dam Design and Construction Manual" is applicable only to dam projects which comply with the following requirements:

1. Project concepts are typical, i.e. entail an embankment, and an earthcut overflow spillway and possibly an operating spillway and/or a gated outlet, and a conventional arrangement of these components.

2. Components are constructed on "good" or "fair" foundations and with "good" or "fair" embankment materials as defined by specific criteria.

3. Projects fall within a specified size classification.

4. Projects fall within a specified hazard potential classification.

1.2.1 **Project Concept**

The "Small Dam Design and Construction Manual" is applicable only to small storage dams and backflood irrigation schemes on natural coulees, small intermittent tributary streams or constructed diversion channels.

Conceptually, a typical small storage dam would include an earth embankment and an earthcut overflow spillway channel. In addition, a small project may have a separate operating spillway and/or a gated outlet structure through the embankment. The earth embankment crosses the valley and retains the stored water. The earthcut overflow spillway channel generally serves as a source of material for the embankment, has an inlet elevation at or marginally above full supply level (FSL), and provides for passage of major flood events. The operating spillway (generally constructed with concrete, rock, steel or wood) provides for passage of watershed base flow and the relatively frequent minor flood events. In some cases a separate structure is not required or warranted for the operating spillway and the earth channel around the embankment discharges all excess flows to the downstream valley. The gated outlet structure provides for controlled releases from the reservoir.

A typical backflood irrigation scheme includes a low embankment across a broad floodplain and an in-channel control structure. The embankment provides a barrier for temporary ponding of a flood event. The control structure limits the discharge rate and water surface elevations.

Projects which are conceptually different or more complex than the two project types described above are beyond the scope of this manual.
1.2.2 **Foundations and Embankment Materials**

The "Small Dam Design and Construction Manual" applies only to projects constructed on foundations and with embankment materials which are classified as "good" or "fair". Foundation and embankment material classification criteria and classification procedures are described in Section 2.5 "Geotechnology". Engineering advice must be sought during the investigation and assessment of a foundation and the construction materials when the designer is unsure of his ability to make an accurate assessment.

Projects which entail a foundation and embankment materials which do not comply with the prescribed criteria should be referred to a qualified professional engineer for project design.

1.2.3 **Size Classification**

The "Small Dam Design and Construction Manual" is applicable only to projects within the following size classification:

1. Maximum dam height measured from the lowest natural ground elevation within the embankment foundation area to the top of dam elevation shall be equal to or less than 8 m.

2. The maximum storage at the top of dam elevation shall be equal to or less than 400 dam$^3$.

All projects which do not meet both requirements should be referred to a qualified professional engineer for project design.

It is noted that these size limits on the application of this manual should not discourage the development of a site potentially attractive and feasible for a larger project. It would be irresponsible to eliminate a potential site from consideration because the most practical embankment would be above 8 m or the storage would be greater than 400 dam$^3$. These dam sites should be studied with the assistance of a qualified professional engineer to determine the most appropriate project for the given conditions and requirements.

1.2.4 **Hazard Potential Classification**

The "Small Dam Design and Construction Manual" is applicable only to projects with a low hazard potential rating with respect to impacts of a hypothetical dam failure. Hazard potential classification criteria and procedures are defined and described in Section 2.2 "Hazard Potential". Projects which do not meet the low hazard potential criterion should be referred to a qualified professional engineer for project design.
1.3 **DESIGN RESPONSIBILITY**

Successful engineering projects are founded on a sound, proven technical basis (criteria, standards, guidelines, methods) and trained, experienced, conscientious, and competent designers. In order to comprehend and apply the information contained in the "Small Dam Design and Construction Manual", design personnel should have completed an engineering technologist program such as that available at the Saskatchewan Institute of Applied Science and Technology or have the equivalent training based on experience. Design personnel must also have experience in the investigation, design, construction and inspection of dam projects in order to exercise appropriate judgement in determining and assessing design objectives and requirements, classifying foundation and embankment materials, classifying hazard potential, and evaluating pertinent project details.

One individual only should be given the responsibility for the project design, including the definition of project requirements and design inputs; execution of field and office investigations; performance of design computations; preparation of construction drawings and specifications; and supervision of construction. This person should prepare and sign all investigation reports and drawings.

A professional engineer, qualified in the area of dam engineering, must conduct a complete review of the design, drawings and specifications, and approve, under signature, all project designs and drawings. Managers may wish to consider the merits of involving the reviewer during site selection and other stages of the work to assist in its efficient progression.

In the case where a PFRA office holds the responsibility for contract administration and construction supervision, the tendering process must be conducted according to the federal government contract regulations and PFRA procedures.
1.4 **PROJECT LIFE CYCLE**

Most projects resulting from the construction of engineering works follow a typical "life cycle" as shown in Figure 1.4.1. Each portion or item in the life cycle refers to a function or activity which must be properly executed for orderly, efficient, cost-effective project development and performance. The owner of a proposed project should be made aware of his responsibilities throughout the life of the project.

The "Planning Process" involves the project proponent and the designer. At this stage, the needs that are to be met by the project and the concept of the project are identified and developed. Project objectives and requirements are determined. (What is the project expected to do?) Conceptual development proposals and sketches are prepared based on reconnaissance-level investigations and available information for a number of alternate site locations. (How is the project going to work?) A preliminary assessment of the water supply potential is conducted. (Is there enough water to satisfy projected demands?) Project locations with insufficient water supply potential are rejected. A reconnaissance-level investigation would include a review of existing information (aerial photos, testhole logs, maps) and a site inspection to identify site characteristics which may constrain or support a particular development concept and to identify potential downstream impacts. Preliminary costs and benefits are estimated based on experience with other projects and available information on the project under consideration. Project alternatives are ranked to assess the relative viability of each. After discussion with the project proponent, one or two sites are selected for further preliminary engineering investigations.

After the initial planning, if the project appears to be technically feasible from PFRA's perspective and economically viable from the project proponent's perspective, preliminary engineering studies are performed for one or more sites. The information in the "Small Dam Design and Construction Manual" should be applied at this stage of investigation. Preliminary engineering studies would include: topographical surveys of the dam and spillway locations and the reservoir area; a project hazard potential assessment; a geological damsite description; a geotechnical assessment; and a preliminary design, complete with cost estimate. The overall project feasibility can be assessed based on the costs and benefits of the proposed project. This final assessment completes the project planning phase. It is noted that some design activities are included in the planning phase. There is an overlap between planning and design, as preliminary designs and costs (design function) are required for the information of the proponent. Although there is an overlap between the planning and design functions, it is still convenient to treat them as separate functions.

After completing the planning studies, if a decision is made to construct the proposed project, a final design is undertaken. If the project parameters lie within the limitations specified in the "Small Dam Design and Construction Manual", the work would be performed by a technician or technologist, with review by a qualified professional
engineer. The final design, including construction drawings and specifications, is completed, documented and approved for submission to the pertinent provincial water resources agency, if this is required prior to construction.

Construction of the project may be undertaken by a contractor or the project owner (proponent) using his own or leased equipment. The extent of PFRA responsibility for ensuring the work is built according to the approved drawings and specifications is dependent on the degree of involvement during construction as provided for under the particular program or agreement. In the case of a project built under the Rural Water Development Program the owner must construct the project according to the approved drawings, specifications and procedures as a condition of the licence and the grant, and should ensure this by using appropriate technical supervision. PFRA field staff are available to provide advice and information.

As-Constructed drawings and an operation and maintenance manual should be prepared by the project designer for the owner. Operation, maintenance and routine inspection of the project during its life is the responsibility of the project owner. Information and advice on these matters, including use of the operation and maintenance manual, is provided to the project owner.

A Project Flow Chart is illustrated in Figure 1.4.2. The flow chart, in this case for the Rural Water Development Program, summarizes various steps for determining preliminary project feasibility, assessing the applicability of the "Small Dam Design and Construction Manual", obtaining project proponent input, implementing project investigations, conducting design studies, and providing advice on project construction, operation and maintenance.
Figure 1.4.1 - Project Life Cycle
2.0 INVESTIGATIONS

2.1 GENERAL

Before preliminary or final designs can be performed, specific information on the project background, project requirements, and site characteristics must be obtained. Investigations are conducted to provide the necessary information for a design. If errors or poor judgment occur in the investigations, the design based on these investigations may be inadequate. All investigations should be documented, reviewed and filed for future reference.

The project investigation items described in this section include: hazard potential; geology; hydrology (water supply potential and flood potential); geotechnology; surveys and mapping. A number of these items (hazard potential; geology; and geotechnology) while necessary for project design also serve as a mechanism for defining the applicability of the manual.

The hazard potential of a project serves to classify the project relative to the consequences of a hypothetical dam failure. It also assists in providing a perspective for selecting standards and criteria to be used in project design.

Knowledge of the site geology assists in assessing the suitability of the site for construction of a dam and the geotechnical parameters and standards to be used in design. Geotechnical information identifies and describes the foundation and soil materials available for construction.

Hydrological investigations provide the information necessary to determine whether the water supply capability of the project drainage basin can satisfy the project requirements, and provide the information for determining project design flood flows.
2.2 HAZARD POTENTIAL

An assessment of the hazard potential from a hypothetical dam failure is required to determine if the proposed project meets criteria which permit the design to be performed by technicians and technologists following the "Small Dam Design and Construction Manual". The Hazard Potential Assessment is performed and documented by the project designer and becomes one of the necessary design inputs. If the assessment identifies a hazard potential greater than the limitations specified in this section, project design is beyond the scope of the "Small Dam Design and Construction Manual".

The criteria, methods and definitions associated with Hazard Potential in this manual have been derived from the PFRA Dam Safety Program (Hazard Potential Reassessment Guidelines and Operation and Maintenance Manual guidelines, March, 1987).

The hazard potential classification of a project is a relative measurement of the potential human and economic losses which may occur as a direct result of a hypothetical dam failure and its associated downstream flooding. It depends on the physical size of the project (i.e. height and storage capacity) and the downstream habitation and environment. The potential losses are assessed using three categories: loss of life by flooding; direct downstream losses by flooding; and other economic losses. Table 2.2.1 lists the three categories and the monetary range assigned to the losses in the second and third categories. The loss of life category is also quantified. The values related to all categories are only general guidelines and judgment is required when differentiating between the various classifications. The hazard potential is taken as the highest classification from each of the individual categories.

2.2.1 Methods for Determining Hazard Potential

In order to conduct a hazard potential assessment, methods are required to estimate the magnitude of flooding from a potential dam failure; to identify the downstream losses; and to evaluate or quantify the downstream losses.

There is no straight-forward universal standard method for assessing hazard potential. In practise the choice of method is a function of the risk and consequences of failure; complexity of problem; and availability of data and resources. Computer simulations and special costing studies may be used for large, complex, high-risk projects located in urban areas. Less sophisticated methods are available for assessing smaller projects located in small urban and rural areas. Projects normally considered for application of this manual are relatively small, simple projects located in a rural setting away from costly works or facilities. In these cases, relatively crude methods, based mostly on observation and judgment, provide satisfactory results for determining the hazard potential.
For application of the method described below, the following information is required: a 1:50 000 NTS map; aerial photographs; approximate dam location; and estimates of dam height and reservoir storage.

Various facilities such as granaries, storage sheds, houses, etc. are identified and marked on the 1:50 000 map, as well as other items which may affect or be affected by a potential failure.

A field inspection of the area downstream of the proposed dam is conducted for a downstream distance of at least 4 km or to where the potential flood wave would be contained within the stream channel. The various facilities identified on the map are confirmed and other facilities discovered during the field inspection are identified and documented on the map. The relative elevation between the adjacent stream bottom and these facilities is estimated. A judgment is made whether a particular item would be flooded by a potential dam failure.

It is noted that if the contents of the reservoir can be contained in the volume below and between the dam and the location under consideration, flooding from a dam failure is unlikely. No flood inundation mapping is required in this method. Flooding and associated consequences and costs are assessed solely by interpretation and judgment of the investigator in the field. Insurance rates and real estate trends may be used to assist in assessing flood damage. However, all costs must be reduced to 1980 dollars using appropriate cost index factors in order to utilize the classification system described in Table 2.2.1. It is noted that for purposes of this manual, it is necessary only to determine if the project hazard potential is "low" (C) or if it is greater than (C). Information for consideration of a "significant" (B) or "high" (A) is not required. When assessing "loss of life" it is usually considered that "loss of life" could occur when flooding greater than 1 m occurs in the first floor of a main residence.

If the investigator encounters a situation which is complex or if he/she is unsure of the applicability of this small dam hazard potential method to a particular project, advice or direction should be obtained from a qualified professional engineer.

2.2.2 Hazard Potential Assessment

All information for determination of the project hazard potential should be recorded and filed at the designer's office. The hazard potential study is summarized and documented in a standard form "Hazard Potential Assessment Summary", whether the assessment was conducted by a technician or a qualified professional engineer. Refer to Figure 2.2.1. The assessment summary is required as one of the necessary design inputs. If the hazard potential rating of a project is not "low" (C) the project design is beyond the application of the "Small Dam Design and Construction Manual".
<table>
<thead>
<tr>
<th>Hazard Potential Classification</th>
<th>Loss of Life by Flooding</th>
<th>Downstream Economic Losses by Flooding</th>
<th>Other Economic Losses</th>
</tr>
</thead>
<tbody>
<tr>
<td>High (A)</td>
<td>more than few</td>
<td>excessive (extensive community, industry, or agriculture; losses greater than $1,000,000)</td>
<td>greater than $2,000,000</td>
</tr>
<tr>
<td>Significant (B)</td>
<td>few (in the order of 1 to 5 lives; no urban developments and no more than a few habitable structures)</td>
<td>appreciable (notable agriculture, industry, or structure; losses in the order of $100,000 to $1,000,000)</td>
<td>$200,000 to $2,000,000</td>
</tr>
<tr>
<td>Low (C)</td>
<td>none expected (No permanent or seasonal structures for human habitation)</td>
<td>minimal (undeveloped to occasional structures or agriculture; potential losses expected to be less than $100,000)</td>
<td>less than $200,000</td>
</tr>
</tbody>
</table>

**Notes**

1. Magnitude of economic losses is in 1980 dollars and is only an approximate guideline. Applicable cost indices are to be applied to the above table for the project under consideration as required.
2. Other Economic Losses include replacement costs and benefits lost until the structure is repaired.
3. Table excerpted from 1980 "Report of the PFRA Dam Safety Committee".
# HAZARD POTENTIAL ASSESSMENT SUMMARY

<table>
<thead>
<tr>
<th>Project Title / Owner:</th>
<th>Location:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Designer:</td>
<td>Date:</td>
</tr>
</tbody>
</table>

**Downstream Schematic Map**

<table>
<thead>
<tr>
<th>Estimated Embankment Height:</th>
<th>m</th>
<th>Office Studies:</th>
<th>yes</th>
<th>no</th>
</tr>
</thead>
<tbody>
<tr>
<td>Estimated Storage at Top of Dam:</td>
<td>dam$^3$</td>
<td>Loss of Life:</td>
<td>yes</td>
<td>no</td>
</tr>
<tr>
<td>Reference Map:</td>
<td>yes</td>
<td>no</td>
<td>Estimated Flood Damage (1980 $):</td>
<td></td>
</tr>
<tr>
<td>Date of Field Inspection:</td>
<td></td>
<td>Estimated Other Damage (1980 $):</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Discussion with Project Proponent:</td>
<td>yes</td>
<td>no</td>
<td>Hazard Potential Rating:</td>
<td>C</td>
</tr>
</tbody>
</table>

*Figure 2.2.1*
2.3 GEOLOGY

The cost-effective exploration of the foundation, borrow materials and any geologic/geotechnical hazard at a small dam project requires the delineation of the site and reservoir geology. To that end, the Geology Section of this manual describes the origin and other geologic attributes of materials normally encountered in the Canadian Prairies and Foothills. The information is presented in two parts. The first part describes the physical characteristics of the most common materials and discusses origin, composition, colour, texture, and weathering, which are common associations to be expected in field exposures. Each material is also described qualitatively with respect to its geotechnical significance or any geological hazard to the extent it could bear on design and construction.

The second part depicts the common field relationships among the geological materials described, and sets out the spatial distribution and orientation in terms of deposits or units which enables them to be recognized, mapped and interpreted in a practical manner. The field relationships are also illustrated by diagrams to assist in interpretation, to demonstrate scenarios of occurrence which are characteristic of the deposits, and to describe geometry of structure where important. The second part is concluded with an overview of typical errors of interpretation resulting from the limited field exposure, or by the often similar appearance of some materials when encountered in isolation.

Both site and reservoir must be explored and studied by the designer to identify, understand, and appreciate the geological setting and any geological hazards. Exploration and evaluation of the geology is essential to an efficient geotechnical assessment and should be completed before or done concurrently with the geotechnical work. In many cases entailing geologic/geotechnical hazards, the geologic assessment alone will be sufficient to prove a site or reservoir is not technically feasible. Guidelines on investigation methodology eg. location and depth of test holes, are given in Subsection 2.5, Geotechnology.

2.3.1 Classification and Engineering Characteristics of Geological Materials

The common geological materials encountered in the southern Canadian Prairies and Foothills are listed and briefly described below with respect to their significance to a small dam project. The classification in the PFRA - Modified Unified Soil Classification System is given when it would be meaningful.

1. **Clay Shale (CL, CI, CH)**

-Material consists of low to highly plastic clay of Cretaceous marine origin.

-In the unweathered, undisturbed state, clay shale is a massive, featureless, intact hard clay with very few joints, i.e. say on 3 m spacing, but it is found in this state only in very deep
excavations below ground level. It weathers readily and to substantial depths, and in the most weathered state at the surface, it is highly fissured or disaggregated, very soft and compressible.

-Color is very dark gray to brownish gray in both weathered and unweathered moist states, and light to medium gray or brown in weathered dry states.

-Materials associated with clay shale include sandstone, bentonite layers, horizons of concretions, and fossil zones. Deposits associated with weathered shale include weathering products such as gypsum crystals (CaSO₄.2H₂O) and thin veins of melanterite (FeSO₄.7H₂O) separating the fragments of shale.

-The most widespread formation of clay shale on the Prairies is the Bearpaw Formation; it underlies glacial deposits or outcrops along the flanks of major valleys over extensive areas in Alberta and Saskatchewan and in Manitoba west of the Manitoba Escarpment. Clay shale strata are also present within the following formations: Lea Park; Riding Mountain (Millwood Member only); Horseshoe Canyon; Foremost; Ashville; Morden; and Ravenscrag.

-Depending on its degree of weathering, clay shale is found in any state ranging from impermeable and incompressible for all practical purposes, but with a potential for substantial swelling, to semi-pervious and significantly compressible. Strength in the short term ranges from very low to very high depending on the degree of fissuring, natural moisture content, and presence of shear zones. Weathered materials are highly erodible. Over the longer term, swelling and other rebound behaviour will reduce the strength of unweathered material exposed in deep cuts or excavations.

-Shales can be hazardous primarily because of the possible presence of very weak, sheared horizons developed through glacial thrusting/drag or rebound following natural unloading or excavation, their potential for swelling and settlement, and their vulnerability to erosion. Slumping of valley sides, is usually evidence of the presence of clay shale at shallow depth.

2. Bentonite (CH)

-This material is highly plastic clay derived from the chemical alteration of volcanic ash. On the Prairies, it occurs only in upper Cretaceous shales and bentonitic sandstones, either mixed with other kinds of clays or as discrete layers.

-Layers range from 5 mm to 500 mm in thickness and, where undisurbed by geological processes, can be traced by drilling for hundreds of kilometres.

-In the unweathered state, bentonite is very hard, brittle, waxy-looking, and yellowish gray, however, it is found in this
state only at substantial depths. In the weathered, moist state it is very soft and sticky, and yellowish greenish gray or occasionally bluish gray in colour. In the weathered, dry state, the material reverts to a granular structure, entailing angular fragments in the medium to large sand sizes.

Bentonite is one of the most hazardous materials in geotechnical engineering in the Prairies because of its low shear strength and very high potential for volume change, i.e. swelling and shrinking, due to wetting or drying.

3. Sandstone

-Sandstone is of Cretaceous or Tertiary shallow-marine, estuarine or littoral zone origin, and is usually composed of uniform, fine to medium-grained, poorly cemented to well cemented sand. Sand texture ranges from well graded to poorly graded with non-plastic fines through to plastic fines. Depending on type and amount of clay content, the sandstone may exhibit considerable plasticity in the Soil Classification System tests.

-Colour of unweathered sandstone is medium gray; weathered outcrops range between pale yellowish gray to pale greenish gray.

-Depending on density and cementing agent, sandstone may be very hard with very high shear strength, or very soft and friable. A dry, low-density poorly cemented sandstone, e.g. with an argillaceous (clay) cement, can undergo short-term settlement upon saturation. Permeability is dependent mainly on the amount of fracturing, to a lesser extent on the gradation and amount of any intergranular material, including any cementing agent. The degree of erodibility depends upon its density and cementing agent.

-Sandstone formations and other formations containing substantial sandstone in the Prairies and Foothills include the Judith River (formerly known as Belly River), Paskapoo, Porcupine Hills, Bearpaw (certain members only), Eastend, Frenchman, and Ravenscrag.

-Dense, well-cemented sandstone constitutes a very competent foundation material. A hazard potential for settlement or erosion exists if the sandstone has a low density and is poorly cemented. Joints and fractures may result in significantly higher local permeabilities which may cause seepage problems.

4. Siliceous Shale and Other Cemented Shales

-The Odanah member of the Pierre Formation is a siliceous shale of special significance in the western prairies. It is found outcropping and at shallow depths in eastern Saskatchewan and in the Manitoba Escarpment and western Manitoba. It is mainly a chemical precipitate with minor clay and silt content.
This shale does not physically soften during weathering but readily disintegrates through cracking and splitting.

It is a pale greenish gray colour with a significantly lower density relative to other shales.

Other shales, chemically cemented by siliceous or calcitic agents, depending on their degree of cementation, range up to hard to very hard and rock-like, generally weather to shallow depth only, and generally constitute a very competent foundation material. Fractured or fissured zones may have significantly higher local permeabilities.

It should be noted that shale from the Odanah member is found up to cobble size in sand and gravel deposits in eastern Saskatchewan and in Manitoba close to outcrop locations along the Manitoba Escarpment, and is recognized as a deleterious material in these deposits with respect to its suitability for concrete aggregate.

5. Preglacial Sand and Gravel (GW, GC, GM, GP)

These materials are important because of their wide-ranging distribution along the floors of valleys that existed prior to the first glaciation.

The texture is often similar to other local and more recent deposits of sand and gravel, but the degree of roundness is much greater. The density is much greater as the deposits have been overrun and compacted by at least several glaciers. Gap-graded deposits are not uncommon, and local permeabilities can be very high. Many outcrops occur as low cliffs or ledges of calcitic cemented zones, but these zones extend only a few metres back into the slope.

They occupy a unique stratigraphic position (i.e. above Cretaceous and Tertiary strata and below glacial deposits).

These granular materials are incompressible and generally exhibit very high shear strength. They may have a high hazard potential in a dam foundation and reservoir due to their high permeability.

6. Till (CI, CL, CH, ML, MH)

This material, also known as glacial clay or boulder clay, is a dense, well-graded, mixture of silty clay with sand, gravel, cobbles and boulders. Deposits can include fragments to large blocks of Cretaceous strata (usually shale or sandstone). Erratics, which can exceed 10 m across, could be considered as a constituent but are encountered very rarely.

Till was deposited from the basal zone of an advancing glacier or left as a residual material following melting of the ice. It
was incorporated into a glacier through scouring of surficial strata and mixing during transport.

-Variations in texture occur and usually reflect the composition of the terrain over which a glacier advanced. Basal till zones containing an increasing proportion of the underlying material usually have a gradational contact with it. In other cases the contact is sharp and well defined.

-Till can range from a compact, massive structure with few joints, to a highly fissured mass of dense fragments. Deposits entail both oxidized zones (brownish gray) and non-oxidized zones (medium to dark gray).

-Till generally represents a competent foundation and a highly suitable construction material, but poorly graded zones or stratified zones of clay, silt, sand or gravel may cause problems; often cobbles and boulders cause difficulty in excavations.

7. Glacio-Fluvial Deposits (SM, SP, SW, GP, GW)

-This material includes a wide range of sand, gravel and cobbles deposited by glacial meltwater downstream of the ice front or in an ice-contact position. It is highly variable in texture and structure, and in its relations with adjacent deposits. The material as defined herein includes deposits of early postglacial age as well as interglacial deposits overlain and underlain by till.

-This material is incompressible, with a high shear strength. Permeability ranges greatly and is dependent on the fines content. The material may be hazardous in foundations because of high permeability. Sandy deposits are erodible.

8. Glacio-Lacustrine Deposits

-Glacio-lacustrine deposits include clays, silts, and fine sands. The most common material is glacio-lacustrine clay, found in glacial lake basins, well beyond the shore zone. Glacio-lacustrine silt and fine sand are found in the foreshore and deltaic zones. Glacio-lacustrine deposits do not include the lag deposits of cobbles and boulders often lying along wave-cut shorelines developed on till.

(a) Glacio-lacustrine Clay (CI, CL, CH)

-The structure of this material ranges from massive and dense with few joints to highly fissured, and texturally from very uniform, with little if any stratification, to "varved clay" comprising highly stratified interbedded clay and silt layers. Ice-rafted debris of pebbles, sand lenses and masses of till is occasionally found.
These deposits are soft to medium-hard depending on the clay content, moisture content, depth and degree of consolidation. Colour is dark gray.

Glacio-lacustrine clays are relatively impervious materials, with low to medium shear strength. Weathered zones may be highly erodible. They can have a very high potential for swelling or settlement depending on their in situ condition and their degree of loading or unloading upon construction.

Deposits found between till deposits are invariably badly sheared with many slickensides, and constitute a very treacherous material of low shear strength. Slumping of valley sides may indicate presence of this material.

With respect to their use as a construction material, their suitability is usually dictated by their natural moisture content, and very wet or very dry clays can be quite unsuitable.

(b) Glacio-lacustrine Silt (ML, MH)

The structure of this material ranges from massive and poorly stratified to highly stratified or varved. It is usually weakly cemented with a low density, making the material very friable.

The colour is medium gray below water table and light brownish gray in dry state above the water table.

Glacio-lacustrine silt is a low-permeability deposit with moderate to low shear strength and moderate to low compressibility. It is highly erodible in nature. Depending on its location, it can be highly undesirable for a dam foundation due to its susceptibility to rapid erosion and potential for piping. Dry, low-density silts may "collapse" upon saturation and undergo significant settlement.

(c) Glacio-lacustrine Sand (SM, SP)

This material is much the same as glacio-lacustrine silt with respect to geologic characteristics, except it is coarser in texture and is seldom found in varves with clay.

Glacio-lacustrine sands form the main source of dune sand.

With respect to small dam design, the material can be considered incompressible, with a moderate to high shear strength, and permeability can range widely depending on fines content. The material can be undesirable for dam
9. Lag Deposits

-This material is an accumulation of well-rounded cobbles and boulders representing lag remnants of prolonged erosion, mainly by water (occasionally by wind) of till. The materials are mainly granite, or metamorphic or very durable sedimentary rock. The most common, readily visible deposits lie along the glacial meltwater channels eroded into till.

-Horizons of lag deposits separating two till sheets are not uncommon; spacing between cobbles/boulders is usually in the order of 1 m to 10 m.

-Lag deposits are generally very resistant to erosion because of the size and durability of the materials, and the deposits are invariably sought after as sources of riprap.

10. Channel Deposits (SM, SP, SW, GP, GW)

-These materials are commonly sand or gravelly sand deposited along the floor of a present-day channel or on the flank of inside bends (point bars). Deposits are highly variable in texture because of the large range in depositional conditions. Deposits of creeks tend to be dirty, i.e. contain considerable fines, which are trapped in the intergranular spaces during slack-water periods.

-The materials are generally incompressible, with permeabilities ranging from moderate to very high depending on the fines content, and with moderate to high shear strength. Fine-grained deposits are erodible.

-These materials may be hazardous where they entail highly permeable layers or lenses and erodible, fine-grained strata.

11. Floodplain Deposits (CI, CL, CH, ML, SM, OL, OH)

-These materials consist of recent, clay, silt or fine-grained sand deposited during overbank flood flow of present-day streams. They are stratified, uniform in a horizontal direction, and have never been loaded by other materials. High organic soils found in abandoned channels on floodplains are included in this class of materials.

-The materials are compressible, with a wide range of strength and permeability.

-They may be hazardous in foundations because of their low shear strength, compressibility and erodibility, and their susceptibility to piping.
12. Dune Sand (SP, SM)

-These materials are loose, well-sorted, uniform, fine to medium-grained sand, transported and deposited by eolian (wind) processes. There is a complete absence of clay sizes and sizes greater than medium sand, although deposits may contain buried organic layers that include organic clay. Deposits are coarsely banded with many truncated boundaries.

-Dune sand is light gray to tan when dry and dark gray to bluish gray below the permanent water table.

-With respect to small dam design, dune sand can be considered to have a low compressibility and a high shear strength. It is highly pervious and highly erodible.

-It can be hazardous for foundations because of its high permeability and erodibility.

13. Slump Debris

-Slump debris results from the failure (landslide) of a valley wall. Slump debris is very difficult, if not impossible, to recognize from samples. It is one of the most treacherous materials from a stability point of view as it is always underlain by a well-developed failure surface which may have a very low shear strength, and there is risk of further movement.

-While there is no question that one would not consider a site where there is evidence of present-day movement or very recent movement, there is always the question whether debris that developed many years ago and has the appearance of being stable is presently creeping or whether the natural conditions that gave rise to the initial movement have diminished only slightly and may reappear.

-Shale slump debris and clay slump debris are far more common than any other kind but every kind of material can be disturbed by slumping if that material overlies the weak zone giving rise to the slumping.

-Overlying clayey materials can be highly fissured and fractured as a result of slumping. Occasionally slickenside material or thin zones of slickensides will be present within shale or clay debris. Original stratification of a deposit may be badly disturbed by slumping. The degree of disturbance and the underlying failure surface can be best recognized in large outcrops or long backhoe cuts.

-Slump debris is usually underlain by a continuous zone of slickensided clay material of very low strength; this zone can range down to 2 or 3 mm in thickness. In most cases the slickensided material will be derived from clay shale, bentonite, or glacio-lacustrine clay.
Extreme variability in the degree of disturbance of slump debris makes it difficult to summarize its engineering characteristics, but it should be noted that the behaviour of slump debris in situ will likely be governed by the strength of the underlying shear zone and piezometric levels along the base of the shear zone.

Depending on its location in the floor of a valley, slump debris may be acceptable in the foundation for a small dam, but the matter would require a focused geotechnical assessment. Excavation into the toe of a slope developed on slump debris is extremely hazardous in that it would likely lead to re-activation of movement.

14. Slopewash (CI, CL, CH)

-This material consists of silty to sandy clay eroded from steep upper slopes and deposited at the toe. Slopewash is a recent deposit and will often grade into floodplain deposits with no well-defined contacts. These materials are coarsely stratified with significant vertical variation, oxidized and brownish-gray in colour, and usually include organic-rich beds.

-Slopewash is soft where not consolidated by drying. It can be highly compressible, erodible, low to moderately permeable with low to moderate shear strength. It is usually quite erodible because of its texture and low density.

-Its high compressibility and low shear strength upon saturation may make slopewash undesirable for a dam foundation.

15. Drift Soil (ML, SM)

-This material is a highly variable mixture of clay, silt, and fine sand with a high organic content. It is common along the flanks of valleys traversing areas where soil drifting is prevalent.

-Depending on its organic content, the material is moderately compressible, moderately pervious, highly erodible, with a low to moderate strength. The compressibility and erodibility make this material hazardous in a dam foundation.
2.3.2 **Common Relationships Among Geological Units**

Site explorations should consist of a number of point observations derived from test holes, backhoe cuts or outcrops scattered around a site. Results from such explorations usually show that a variety of materials is present at every site. These materials may have some common characteristics, or they may not have any. There may be materials with similar plasticity but different strengths, or similar textures but different permeabilities. Some materials may be advantageous with respect to foundation conditions or usage for borrow. Other materials may be quite disadvantageous from these points of view. Clearly, it is necessary to organize these observations to develop an accurate and complete picture of soil conditions over the entire site. The most practical approach by far is to classify the site materials into geological units, i.e., according to their geological origin.

For purposes of exploring and evaluating a site for a small dam, site materials can be classified into two major groups: Pre-valley units, major and minor, and Valley units. These groups are illustrated in Figure 2.3.1.

The pre-valley units are deposits that were laid down prior to development of a valley. The pre-valley units usually extend horizontally well beyond the sides of the valley and they also underlie the valley at some depth. They predate the valley, possibly by several hundred years or maybe by several million years. In any case, the valley has had no influence whatsoever on their origin and on their location. However, as noted in the following sections, a pre-valley unit could be involved in some geologic process associated with development of the valley, in which case the material would lose its original classification and be considered to form or lie within a valley unit.

The valley units are those that have resulted from or are related to development of a valley and, in most cases, these are very recent deposits compared to the age of the pre-valley units. In very rare cases, where there have been erosional processes only along a valley, i.e. no depositional processes, there will be pre-valley units present, but no valley units.

1. **Major Pre-Valley Units**
   a. **Preglacial Strata (Tertiary, Cretaceous and older)**

Units which are the oldest, extend to a greater depth than any other, and which are always found underlying a site at some depth are the preglacial strata units. Over most of the Prairies and Foothills these are shales of a variety of Cretaceous and Tertiary formations; less common are sandstone formations and sandstone units lying within the shale formations. In central and eastern Manitoba, limestone and dolomite are encountered; in the Cypress Hills in Saskatchewan and Alberta the units may include a conglomerate of sand and gravel. The shales can be highly weathered, fractured and soft, and the
conglomerates may have no cementing agent. These strata may be very difficult to distinguish from very recent deposits of clay or sand and gravel, and they may pose similar or worse geological/geotechnical hazards. As the term "bedrock" can imply different things to different people it is not used in this manual, to prevent passing along any false inference of competence or suitability with respect to foundations or borrow materials.

These preglacial units may be found exposed on the upland adjacent to a valley. They can be found exposed in the lower flanks of a valley or they may be deeply buried by a thick mantle of younger materials and not be an important factor at all in the evaluation of a damsite.

Shale, sandstone, limestone, dolomite and conglomerate units are the common preglacial strata units. Less frequent units are coal and bentonite. All of these units are sedimentary in origin, and the normal expectation is that the units as well as the bedding lie horizontally. Such is far from being the case in many areas of the Prairies and Foothills. The original attitude of the bedding can be disturbed by any one of three distinct geological processes, as illustrated by Figures: 2.3.2; 2.3.3; and 2.3.4.

i. mountain-building forces

These forces have given rise to the major folds and faults found in the Foothills in Alberta. The strata as well as the faults may have dips well over 45 degrees and some faults are tens of metres in thickness. Fortunately most of this disturbance is limited to a region of relatively strong rock, and although the rock is badly broken in many areas, the consequence of this disturbance with respect to the required strength of material at a site for small dam is not significant. However, the permeability of a fault may be significantly high and if a major fault lies in the abutment at a site or lies under the reservoir, large seepage losses could occur.

ii. salt-solution collapse

The Prairie Evaporite Formation, which contains the major potash deposits of Saskatchewan and lies several thousand metres below the surface, is not present everywhere, and it is clear from deep exploration programs that in certain areas the beds have been dissolved over geologic time by migrating groundwater. This has resulted in the collapse of overlying strata. There is only widely scattered, local evidence of movement in post-glacial times and there is no meaningful risk to small dams from any present-day movement or future movement that might be anticipated. During the collapse, the overlying strata were broken into large
fault blocks estimated to measure several hundred metres across, and limited data suggest these fault blocks are separated by faults measuring less than one metre in thickness. Dips of the strata within the fault blocks are usually low, i.e., less than 10 degrees, whereas dips of the faults separating the fault blocks can range between 45 degrees and 90 degrees. The faults are formed of tightly packed, slickensided clay gouge. It is highly unlikely that faults will be encountered at sites for small dams on the Prairies, however, wherever they occur, they interrupt the continuity of the strata and they may present a very hazardous condition in that the faults have a very low shear strength and may represent potential slip surfaces.

**iii. Glacial Disturbance**

The advance of the glaciers across the Prairie disturbed the upper zones of shale and sandstone in many areas, particularly where these materials lie at the toe of northeasterly facing slopes. There are only a few exposures to adequately show the nature of these disturbances. It is clear that the effects include intense fracturing of large masses of material, the deformation of large masses into folds, and the faulting of strata across the bedding as well as along the bedding, especially along any bentonite layers which may be present in the shale units or along contacts between strata. Deformation can be recognized only if marker beds or some evidence of the original bedding is present. A bentonite layer in shale represents a reliable marker horizon provided there has not been shearing along the layer. Very good examples of glacial disturbance are found at Claybank, east of Halbrite on the Souris River Valley, and south of Tantallon on the Qu'Appelle Valley.

**b. Till**

The shale, sandstone and other units mentioned above are often directly overlain by till. Till deposits extend horizontally over great distances. The contact with an underlying unit is usually gently undulating. As there have been several glaciations it is not uncommon to have more than one till unit present at a site. The engineering characteristics of different till units are usually similar. Differentiation between units is sometimes difficult and usually not important with respect to small dams design. As for the shale and sandstone units, till may be found exposed on the upland adjacent to a valley. It may be found forming the flanks of the valley, or it may lie deeply buried beneath a thick mantle of younger materials and may not be an important factor at the site. However, where slumping has occurred
along a valley, till materials may have been involved and transported down the slope. In this case the till materials should be considered a valley unit and mapped in with the slump debris.

c. Glacio-Lacustrine Units

Glacio-lacustrine clay, silt and sand, if present at site, usually overlie till. These are deposits of extensive glacial lakes and, as such, are also very extensive in their horizontal extent. Many of the larger glacial lake deposits on the Prairies entail a lower unit of glacio-lacustrine silt overlain by an upper unit of glacio-lacustrine clay. Bedding of these units and the contacts with the underlying units is usually horizontal. Whenever present, these units usually form the most recent and the uppermost upland deposit in the vicinity of a valley.

Thick deposits are found in the major glacial lake basins, of which glacial Lake Agassiz, glacial Lake Regina and glacial Lake Rosetown are examples.

2. Minor Pre-Valley Units

These deposits are of minor importance only with respect to the frequency of their occurrence in valleys located on the Prairies. However, where they occur at a damsite, they can present problems and in some cases, very hazardous conditions and they should not be ignored. Some possible relationships with major pre-valley units shown in Figure 2.3.5.

a. Preglacial Sand and Gravel

These are generally conceived to be the deposits of major rivers that flowed across the Prairies and Foothills prior to the first glaciation. As such they occupy a unique position in the geological column. That is, they overlie the shale, sandstone, limestone, dolomite or conglomerate units and they underlie the lowermost till. As the preglacial river valleys were very broad in width, ranging up to 20 km across, these deposits may be present over relatively large areas. Thicknesses as much as 50 m have been noted.

b. Intertill Glacio-Lacustrine Clay

Most geological deposits laid down during interglacial periods appear to have been destroyed during the advance of the subsequent glacier and there is little present-day evidence of their past existence. However, remnants of glacio-lacustrine clay, in particular, do occur between till units, and although they are not common by any means, they are invariably badly sheared with considerable slickensided material present. As such they can pose very hazardous stability conditions at a damsite. Highly slickensided, intertill glacio-lacustrine clay was noted in vicinity of the
Shellmouth Dam on the Assiniboine River and the Paddle River Dam northwest of Edmonton.

3. Valley Units

These units are products of the development of a valley or development of a valley has brought about their deposition in the valley or in the immediate vicinity. As such they are restricted to the valley flanks and the valley floor. Their location, dimensions, and relationship to other valley units as well as to the pre-valley units are of utmost importance, and they form the focus of the exploration and geotechnical assessment activities at most sites. It is essential that the exploration clearly distinguish between these units and the pre-valley units.

a. Lag Deposits

Whenever present, these are usually the oldest of the valley deposits. Nevertheless, they do not always lie in the lowermost position in the valley cross-section. The deposits are found mainly lying along the floor and flank of major glacial meltwater spillways incised in till, and as these streams are considered for purposes of this manual as a forerunner of the present-day drainage, they are included as a valley unit. They are also found buried below floodplain deposits lying on the contact with underlying till. Fluvial erosion has removed all grain-sizes found in the till other than the cobble and boulder sizes. The deposits are usually not thick and the thickness often ranges about the diameter of the larger boulders in the deposit. The deposits adjacent to the Souris Valley between Weyburn and Estevan, the deposits adjacent to the Arm River in the vicinity of Craik and Chamberlain, and the deposits adjacent to the Moose Jaw River Valley downstream of the city represent some of the larger lag deposits on the Prairies.

b. Glacio-Fluvial Deposits

For purposes of this discussion, the unit is assumed to include deposits of glacial meltwater that once flowed down the valley during the early stage of valley development as well as deposits of subsequent flows during later stages of valley development. Experience has shown that on the Prairies these deposits are usually much coarser than the floodplain and channel deposits of the streams that presently occupy the valleys and there is usually no difficulty in separating the two in the field.

As indicated in Figure 2.3.6, glacio-fluvial deposits may occur as terraces located well above the present floodplain or they may occur at lower elevations, completely or partially filling the lowermost section of the valley and be overlain or at least partially buried by the channel and floodplain deposits of the present-day drainage.
c. Channel and Floodplain Deposits

These units are the most common valley units and are the depositional products of the present-day drainage. Channel deposits are usually the coarser of the two and are deposited by the stream only within its channel. However, as a stream may change its position with time and migrate back and forth across the valley floor, the channel deposits can have an extensive distribution. In a meandering stream, deposits laid down on the inside bank at the channel bends are known as point bars. In many streams, the pattern of deposition is less consistent, and many deposits are lensy, discontinuous and somewhat erratic in distribution.

Floodplain deposits are laid down on the valley floor during overbank flood flows. Stream velocities across floodplains are usually low compared to the velocities in the channel, and the floodplain deposits are invariably clay, silt, and fine sand. Abandoned channels, filled with organic soils, are often present on the larger floodplains and are specifically identified but included in the floodplain unit for reporting purposes. Common relationships are illustrated in Figure 2.3.7.

d. Slopewash

Slopewash is a very common deposit found along the lower flanks of valleys developed in the clay-rich units, i.e., shale, till and glacio-lacustrine clay. The unit is formed from material eroded from the upper slopes by water and wind and deposited at the toe. Where the stream on the valley floor is sluggish and associated with deposition of finer-grained materials, the slopewash usually grades imperceptibly into the floodplain deposits both texturally and topographically, and it is very difficult to identify a sharp break between the two units. The bulk of a slopewash unit is laid down during very intense rainstorms, and topsoil stripped off the upper slopes is considered to be one of the sources of the buried organic horizons often found in these deposits.

e. Dune Sands

This unit is not common, but where present it can badly confuse the interpretation and identification of other units. In most cases, as illustrated in Figure 2.3.8, the sand is blown in from the upland on one side with the deposit draping and mantling that same flank of the valley. In extreme cases, the sand may have migrated in the form of dunes down the flank of a valley and migrated across the floodplain to the channel.

f. Slump Debris

The lateral extent of this unit on the upper flanks of a valley is easily identified on the basis of its characteristic
topographic expression. However, the lateral extent on the lower flanks and floor of the valley is sometimes not as obvious as the toe of the unit may be buried by more recent deposits e.g., slopewash or floodplain deposits. The fact that the toe of a slump may be buried and therefore interpreted to be an "old" deposit is not assurance the unit does not represent a hazard to a damsie, particularly if excavation is made at the toe. Several relationships that may be found along valleys on the Prairies are illustrated in Figure 2.3.9.

2.3.3 Errors in Interpretation of Relationships

Errors in interpretation are common where the data are limited. However, the frequency of errors can be substantially reduced if, in addition to identifying the origin of each material, the chronological relationships between units implied by their origin are established and checked for compatibility with the spatial position of the units, in particular their vertical and lateral extent and succession. Figure 2.4.10 illustrates some typical interpretation problems.

2.3.4 Geological Damsite Description

Results of the geological investigations performed by the project designer should be summarized in the standard report "Geological Damsite Description". Refer to Figure 2.3.11. The report describes the sources of information, identifies the geological units and hazards at the site and presents an approximate geological cross-section at the damsie location. One or two additional cross-sections depicting the upstream reservoir geological conditions should be included. The Geological Damsite Description will assist in the geotechnical studies for classification of foundation and borrow material.

In situations where the geology of a site appears to be unusual or complex, assistance in mapping the units should be obtained from a qualified engineering geologist. Potential geological hazards identified in the "Geological Damsite Description" will be addressed in the geotechnical investigations. In situations when it is concluded that the geological hazards make the foundation "poor", the "Small Dam Design and Construction Manual" does not apply and the design is referred to a qualified professional engineer.
Pre-Valley Units: A, B, C, D, E, F
Valley Units: I, II, III

Figure 2.3.1 - Two Major Groups of Geologic Units

Figure 2.3.2 - Disturbance Due to Mountain-Building Forces

Figure 2.3.3 - Disturbance Due to Salt-Solution Collapse

Figure 2.3.4 - Disturbance Due to Glacial Thrusting
Figure 2.3.5 - Minor Pre-Valley Units and Their Relationship to Major Pre-valley Units

Figure 2.3.6 - Valley Units: Lag Deposits, Glacio-Fluvial Deposits (Basal and Terrace Locations) and Channel and Floodplain Deposits
Figure 2.3.7 - Channel and Floodplain Units

Figure 2.3.8 - Migration of Dune Sand Down Flank of Valley and onto Floodplain
Figure 2.3.9 - Various Relationships for Slump Debris

Oversteepening of Slope by Downcutting Stream

Dormant Slump, Toe Mantled by Slopewash

Oversteepening of Slope by Lateral Stream Erosion

Dormant Slump, Toe Buried by Floodplain Deposits

Failure of Slope Independent of Present-day Stream

Slump Debris Isolated From Main Body of Slump Debris
Figure 2.3.10 - Some Hypothetical Opportunities for Error in Interpretation
GEOLOGICAL DAMSITE DESCRIPTION - SUMMARY SHEET

Project Title / Owner:  

Location:  

Designer:  

Date:  

Site or Centreline Designations:  

Site comments based on interpretation of the following information:

Field Exploration Data  
Soil Maps and Reports  
Subsurface Investigation  
Local Water Well Logs  
Geological Maps and Reports  
Air Photo Interpretation  

A. Abutments

i) Geological Unit(s) forming abutments: Pre-valley [P] or Post valley [V]
   - Clay  
   - Silt  
   - Sand  
   - Gravel  
   - Till  
   - Shale  
   - Sandstone  
   - Other units present:  

ii) Evidence of abutment instability: None  
   - Dormant  
   - Active  

iii) Evidence of natural seepage: None  
   - Present  

iv) Estimated valley slopes: ____ : 1

B. Foundation

i) Geological Unit(s) immediately underlying valley floor: Pre-valley [P] or Post valley [V]
   - Clay  
   - Gravel  
   - Shale  
   - Silt  
   - Till  
   - Sand  
   - Sandstone  
   - Other units present:  
   - Estimated maximum thickness: _____ meters

ii) Evidence of high water table: None  
   - Present  

iii) Evidence of salt concentration on surface: None  
     - Present  

C. Reservoir

i) Cultural features affected by reservoir
   - Buildings: None  
     - Present  
   - Roads: None  
     - Present  

ii) Evidence of valley side instability: None  
    - Dormant  
    - Active  

D. Materials of Construction

<table>
<thead>
<tr>
<th>Material</th>
<th>Local (within 200 m)</th>
<th>Within 1 km</th>
<th>Unknown or &gt;1 km</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay or Till</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sand and Gravel</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cobble and Boulders</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

E. Schematic Valley Cross-Section at Embankment Centreline (attach additional sheets for reservoir cross-sections)

Figure 2.3.11
2.4 HYDROLOGY

The water supply potential is required in the early planning stage to determine if the watershed is able to supply sufficient water to satisfy project demands. The flood potential is required to select the design flood and size the spillway system. The project designer conducts the hydrology studies to determine the water supply potential and flood potential and summarizes the information in a standard report "Hydrology Summary Report".

The Hydrology Section provides a convenient handbook method for estimating flood peaks and evaluating water supply potential for small prairie watersheds (i.e. 50 km² or less). Hydrological information for larger watersheds can be obtained from a qualified professional engineer. This section was prepared with the following attributes in mind:

1. The handbook section (instructions) should be easy to use and should provide quick but adequate estimates.

2. The assumptions and engineering judgements made in preparing the handbook section should be logical and reasonable.

3. The reasons for flood peak and water supply potential variation from one watershed to another should be readily understood and relate to basic phenomena in the hydrological process.

4. The judgment required by the user should be minimized.

The Hydrology Section has an "explanation" section and a "how to do it" section for both the Flood Potential Subsection and the Water Supply Potential Subsection. The user should read the explanation for each of these subsections so as to understand the relevant hydrological concepts. However, when assessing the hydrologic aspects of a project, the prescribed procedures can be used much like a handbook, with a minimum amount of judgement.

An important hydrological concept that pertains to flood and water supply potential is the concept of drainage areas. For prairie streams, the drainage area contributing to runoff is not constant for all runoff events, but fluctuates from one event to another. From this general concept of fluctuating drainage area, two arbitrary concepts were proposed by Stichling and Blackwell. These concepts are described below.

The **gross drainage area** of a stream at a specified location is that area encompassed by its height of land boundary (i.e. the drainage divide between adjoining watersheds).

The **effective drainage area** is that portion of a drainage area that might be expected to contribute to runoff in an average year. This area excludes marshes, sloughs and lakes that would not spill to
the main stream in an average year, along with their local drainage areas.

In theory, gross and effective drainage area boundaries would appear to be distinct. However, in drawing a line on a topographic map that represents a drainage area boundary, considerable judgement is required, especially for poorly-drained watersheds which are characteristic of the prairies. The gross drainage is bounded by the height of land of the watershed and, in theory at least, the gross drainage area boundary is a definite line on the earth defined primarily in terms of topography. By contrast, the effective drainage area boundary is a conceptual line that encloses an area which contributes to run-off under "average" conditions. Thus, the effective drainage area boundary is somewhat subjective and is defined primarily in terms of hydrologic factors.

A description of other hydrological terms are included in Appendix E - "Definitions".

2.4.1 Flood Potential

2.4.1.1 General Concepts

Ideally, the estimate of the magnitude of flood peaks and their probability of occurrence should be based on a statistical analysis of recorded streamflows at or near the project site. Unfortunately, there are very few small watersheds that have records, and those that do have records do not have a sufficient period of record to adequately define the 1:100 or even the 1:50 flood. Thus, all attempts to estimate flood peaks must, of necessity, be inferred from the limited hydrological data and relevant meteorological data available using mathematical models of varying degrees of sophistication and/or complexity.

The approach proposed in this manual was based on the observed flood peaks for a few small streams, rainfall data, antecedent soil moisture assumptions and considerable engineering judgment. It adhered to the following general assumptions.

1. Extreme flood peaks on small basins will be caused by intense, short-duration rainfall.

2. For a given probability of occurrence (i.e. 1:10 to 1:100 events), the magnitude of the flood peak will depend on the size and drainability of the basin, the amount of short-duration rainfall and the antecedent soil moisture.

The method selected to calculate flood peaks is known as the Index Flood Method. In this case, the 1:2 flood was chosen as the Index Flood. To obtain flood peak estimates for desired probabilities, the Index Flood is multiplied by an appropriate multiplier for the corresponding probability. For example, if the peak flow for the Index Flood (i.e. 1:2 flood) was 2 m$^3$/s and the multiplier for the 1:100 flood was 17, the 1:100 flood peak would be 2 $\times$ 17 = 34 m$^3$/s.
For a watershed of a given size and drainability, the flood peak for a specified probability of occurrence will vary from one location to another throughout the three prairie provinces. Flood peak adjustment factors are embodied in the approach to account for variation in the expected rainfall amounts and antecedent soil moisture conditions throughout the prairie region.

2.4.1.2 Development of Flood Determination Model

The procedure for determining flood peaks was developed based on the consideration of several key elements which are discussed as follows:

a) Index Flood (i.e. 1:2 flood)

Eighteen flow-metering sites were selected for the study to establish the 1:2 flood peaks for small basins. Of the 18 basins investigated, nine were Water Survey of Canada (WSC) stations and nine were PFRA Spring Runoff Monitoring Program stations. Only stations having at least six years of recorded instantaneous peak flows were selected. The average length of record was 14 years. A frequency analysis for annual instantaneous flood peaks was performed for each station to establish the 1:2 instantaneous flood peak for that station. Points representing the 1:2 instantaneous flood peaks and their corresponding effective drainage areas were plotted on a graph of flood peaks versus drainage areas. Envelope curves representing the limits of the 1:2 flood peak for basins ranging from well-drained to poorly-drained were drawn as shown in Figure 2.4.1.

b) Drainability of the Watershed

A very important factor influencing the magnitude of a flood peak is the drainability of the watershed. The drainability is a function of the slope and the shape of the watershed, the development of drainage channels and the amount of depressional storage, the vegetative cover and soil texture. The manual was developed for prairie watersheds; therefore, a well-drained watershed is a relative term that relates to prairie conditions. A watershed that is well drained from a prairie perspective would be judged poorly drained if located in a mountainous or extremely hilly region.

From the data available, only general guidelines can be provided to assess the drainability of a watershed. A well-drained basin is one that has an average slope along the main channel of the watershed of 0.004 m/m or more. The combined area of depressional storage (sloughs) should be less than 5% of the total watershed area. The watershed should have a reasonably compact shape with a well-defined main drainage channel for at least 75% of the length of the basin. An assessment of drainability will usually be made from examination of 1:50,000-scale NTS maps. A field assessment is recommended, especially for the smaller watersheds (i.e. 4 km² or less) to supplement the map assessment.
c) Multipliers of the Index Flood

The method for determining peak flows for selected probabilities of occurrence for a given watershed involves the Index Flood and appropriate multipliers. The relationship of the multipliers to flood probabilities and watershed areas (see Figure 2.4.2) was developed based on engineering judgement, taking into account previous studies and relevant hydrological data from small streams.

Previous studies by Durrant and Blackwell 1961 and Aaston 1986 utilized the Index Flood Method for calculating the magnitude and frequency of floods for prairie streams. However, these two studies were based primarily on flow data for larger streams and are not applicable to small basins, although the studies do provide a frame of reference for comparing multipliers for large basins relative to those applicable for small basins. In developing the multiplier relationships, multipliers for small basins were assumed to be larger than corresponding multipliers for large basins. Furthermore, considerable weight in determining multipliers was given to the maximum recorded rainstorm runoff floods on WSC hydrometric stations Fahlman Creek near Davin (05JF008) and Creighton Tributary near Totnes (05HF014). The extreme flood peaks recorded at these two stations were assumed to be in the order of a 1:100 event.

d) Adjustment Factor for Rainfall

All other things being equal (drainability of watershed, antecedent soil moisture conditions, duration of rainstorm), the flood peak is assumed to be proportional to the magnitude of the rainfall. An adjustment factor to account for the difference in rainfall potential from site to site was developed for this manual. As inferred from the study by Hogg and Carr, the amount of rainfall for a given duration and probability of occurrence generally decreases as the site of interest moves from east to west across the prairies (i.e. the rainfall amount would be higher at Winnipeg than at Lethbridge). The adjustment factor reflects this trend.

To illustrate the range of extreme short-duration rainfall amounts throughout the prairie region, the six-hour 1:100 rainfall amounts were calculated using the procedure recommended by Hogg and Carr. Figure 2.4.3 shows isopleths of the six-hour 1:100 rainfall amounts. Even though the duration of flood-producing rainstorms will be less than six hours for the smaller basins (i.e. 10 km² to 1 km²), it was assumed that the trend of potential rainfall (i.e. rainfall isopleths) for shorter storms would approximate that for the six-hour storm. Thus, the pattern of rainfall isopleths for a six-hour storm was used in developing the flood peak adjustment factors due to rainfall amount, and this adjustment factor was assumed to be applicable to all flood studies. The adjustment factor relates to the location of the study site in the three prairie provinces and accounts for the variation in rainfall potential throughout the prairie region. The adjustment factor applied to a given site can be obtained from Figure 2.4.4 after obtaining the appropriate rainfall amount from Figure 2.4.3.
e) Adjustment Factor for Antecedent Soil Moisture

All other things being equal (drainability of watershed, duration and magnitude of rainfall), the flood peak is assumed to be proportional to the magnitude of the antecedent soil moisture (i.e. the wetter the soil, the greater the flood peak). Ideally, the most likely antecedent soil moisture condition at a given site would be established from a statistical analysis of measured soil moisture conditions antecedent to intense, short-duration rainstorms. Since such data is not available, another indicator was chosen. The average accumulated May-June net evaporation is used as an indicator of the antecedent soil moisture (i.e. the higher the net evaporation, the lower the soil moisture). The most likely date of rainstorm occurrence was assumed to be July. Net evaporations as calculated by the Hydrology Division, PFRA, were used. Figure 2.4.5 shows isopleths of the accumulated May-June net evaporations for the prairies. The adjustment factor applied to a given site can be obtained from Figure 2.4.6 after obtaining the appropriate May-June net evaporation from Figure 2.4.5.

2.4.1.3 Procedures for Flood Peak Determination

- a) Delineate Effective Drainage Area

Delineate the effective drainage area tributary to the study site using procedures recommended by Mowchenko and Meid. Make a field inspection of the study basin, if necessary, to help evaluate the drainage boundary.

- b) Determine Drainability of the Watershed

Determine the drainability of the watershed by studying the relevant 1:50,000-scale NTS maps, supplemented by field inspection when appropriate. The following guide should be used to determine the drainability of a study basin. Five factors are assumed to be significant in determining drainability; namely, basin slope, drainage channel development, basin shape, vegetative cover, and soil type. A brief description of each factor is provided as follows.

1. **Basin Slope Drainability Factor (DF₁):** This factor relates to the general basin slope along the main channel or drainage course.

2. **Drainage Channel Development Drainability Factor (DF₂):** This factor relates to the ability of the basin to remove runoff water from the basin and direct it to the outlet. Drainage channels increase the drainability, while depressional storage decreases it.

3. **Basin Shape Drainability Factor (DF₃):** This factor is an evaluation of the compactness of the drainage basin.

4. **Vegetative Cover Drainability Factor (DF₄):** This factor relates to the extent of vegetative cover as it retards runoff flows.

5. **Soil Texture Drainability Factor (DF₅):** This factor relates to the soil texture and its effect on infiltration and thus on runoff.
A range of numerical values for each factor is provided in Table 2.4.1 "Drainability Factors Selection Chart". Select the value that best represents the influence of the factor for the study basin, interpolating between given values as required. Sum up the values to get an overall drainability factor. For the drainage area of the study basin, the Index Flood can be selected from the curves representing the range of basin drainabilities as indicated in Figure 2.4.1. Most prairie watersheds will fall within the overall drainability range of 0.2 and 1.0.

c) Determine the Index Flood (1:2 Flood)

Select the value for the Index Flood for the effective drainage area and drainability of the study basin using Figure 2.4.1.

d) Select Index Flood Multipliers

Select the multipliers appropriate to the 1:10, 1:20, 1:50 and 1:100 flood events for the basin effective drainage area using Figure 2.4.2.

e) Calculate Unadjusted Flood Peaks

Calculate the unadjusted flood peaks by multiplying the Index Flood value by the appropriate multipliers. The flood peaks calculated to this stage apply to a hypothetical reference site having a specific rainfall and antecedent soil moisture.

f) Determine Flood Peak Adjustment Factor due to Rainfall Amount

The flood peak calculated in item e) must be adjusted to account for the difference in the expected rainfall at the study site as compared to the expected rainfall at the reference site. First, locate the study site on the map of the three prairie provinces (see Figure 2.4.3). Interpolate between isopleths of rainfall to get rainfall at the site. Then use Figure 2.4.4 to select the appropriate adjustment factor for rainfall at the site. This adjustment factor compensates for the rainfall difference between the reference site and the study site.

g) Determine Flood Peak Adjustment Factor due to Antecedent Soil Moisture

The flood peak calculated in e) must also be adjusted to account for the difference in expected antecedent soil moisture at the study site as compared to the expected antecedent soil moisture at the reference site. The May-June net evaporation is used as an indicator of antecedent soil moisture. First, locate the study site on the map of the three prairie provinces (see Figure 2.4.5). Interpolate between isopleths of net evaporation to get net evaporation at the site. Then use Figure 2.4.6 to select the appropriate adjustment factor for net evaporation at the study site. This adjustment factor compensates...
for the difference between the antecedent soil moisture at the reference site and that at the study site.

h) **Determine Adjusted Flood Peak**

To calculate the adjusted flood peak at the study site, take the unadjusted flood peak from item e) and multiply by both the rainfall adjustment factor from item f) and the antecedent soil moisture adjustment factor from item g). This adjusted flood peak represents the flood potential at the study site.

2.4.1.4 **Example of Flood Peak Calculation**

The following example illustrates the procedure required to calculate a 1:100 flood peak for a given project. The project, located at 50° 00' 00" longitude and 108° 00' 00" latitude, has a moderately-drained basin of 20 km² (effective drainage area) as determined from 1:50,000-scale NTS maps and field inspection. The overall drainability factor was determined to be 0.60 based on Table 2.4.1 - Drainability Factors Selection Chart.

The steps required to determine the 1:100 flood peak are listed as follows.

1. Using Figure 2.4.1, read 1.75 m³/s for the 1:2 instantaneous flood peak (i.e. the Index Flood) for a moderately-drained basin of 20 km².

2. Using Figure 2.4.2, read 16.1 as the Index Flood Multiplier appropriate to the 1:100 flood for a 20-km² drainage basin.

3. Calculate the unadjusted 1:100 instantaneous flood peak by multiplying the Index Flood by the Index Flood Multiplier (1.75 x 16.1 = 28.2 m³/s).

4. Using Figure 2.4.3, read 57 mm as the 1:100 six-hour rainfall for the given site location (50° 00' 00" longitude and 108° 00' 00" latitude).

5. Using Figure 2.4.4, read flood peak adjustment factor due to rainfall to be 0.811 for a 1:100 six-hour rainfall of 57 mm. (Retain this factor for future calculations.)

6. Using Figure 2.4.5, read 89 mm as the average May-June net evaporation for the given site location (50° 00' 00" longitude and 108° 00' 00" latitude).

7. Using Figure 2.4.6, read flood peak adjustment factor due to antecedent soil moisture to be 0.863 for an average May-June net evaporation of 89 mm. (Retain this factor for future calculations.)
8. Calculate the adjusted 1:100 instantaneous flood peak at the study site by multiplying the unadjusted flood peak by the two previously-determined adjustment factors (i.e. 28.2 m³/s x 0.811 x 0.863 = 19.7 m³/s). The resultant value (19.7 m³/s) is the 1:100 instantaneous flood peak appropriate to the study site.

2.4.1.5 Uses and Limitations

The "model" applies to small watersheds (i.e. 50 km² or less) that have not been appreciably altered by construction works. If a watershed contains dams, embankments or drains that could appreciably alter normal flows, appropriate adjustments to design floods should be made. These cases should be referred to a qualified professional engineer for assistance.

2.4.2 Water Supply Potential

2.4.2.1 General Concepts

The water supply potential of a basin depends on the runoff volume from the basin and the ability of the user to utilize this runoff. Better control and utilization of the runoff can be obtained through use of a reservoir to carry over runoff water from month to month and from year to year. However, in some instances, runoff water will be diverted directly from the live stream and used for such purposes as backflood irrigation (i.e. without the benefit of a storage reservoir). This manual provides guidelines for evaluating the water supply potential of a basin for both storage and non-storage conditions.

a) Storage Condition (i.e. Reservoir)

Assuming that a storage reservoir is used, the water supply potential of a watershed depends primarily on the following seven factors.

1. Magnitude of Runoff Volume

The median annual runoff at the reservoir site can be used as an indicator of annual runoff volume. The Hydrology Division has developed maps showing the median annual runoff for the prairie provinces. Refer to Figure 2.4.7.

2. Runoff Pattern

The variability of runoff volumes from one year to the next affects the draft that can be obtained from a reservoir. The number of consecutive low flow years is a significant factor affecting the draft of a reservoir. However, the most severe low flow period affecting reservoir draft cannot be determined from a simple observation of the array of annual flows. For example, one cannot tell from observation only whether a period of three consecutive years of very low flows is more severe than a period of six years of moderately low flows.
(The water supply potential of a stream, with its particular runoff pattern, can only be properly assessed by making a water budget simulation.)

3. Reservoir Capacity

The magnitude of the reservoir capacity in relation to the runoff volume has a major impact on the available draft. For convenience in this manual, the design reservoir capacity is expressed as a percentage of the median annual runoff at the reservoir site.

4. Reservoir Efficiency

Because evaporation losses decrease the available draft of a reservoir, an efficient reservoir will have a small water surface area relative to the volume of water stored. Thus, reservoir efficiency increases as the ratio of surface area to volume of water stored decreases.

5. Net Evaporation

Net evaporation varies throughout the three prairie provinces. As the evaporation rate increases, the amount of stored water that is available for use decreases.

6. Reservoir Withdrawal Pattern

Because winter ice decreases the amount of stored water that is available for use during the winter, a summer withdrawal pattern will provide a larger reservoir draft than a year-round withdrawal pattern. A variable withdrawal pattern will also provide a somewhat different draft than a uniform withdrawal pattern.

7. Acceptability of Shortages

Accepting shortages in some of the years allows for a greater utilization of stored water in the long run. As more shortages are accepted, more of the stored water is used when it is available. Thus, less water is lost to evaporation and spillage because less water is carried over from year to year.

From a hydrologic perspective, the optimum storage of a reservoir is that storage beyond which an increase in storage does not produce an appreciable increase in draft. This optimum storage can be determined from a storage-draft curve and is that point in the curve where the curve rises more sharply than previously. Refer to Figure 2.4.13. There is usually not a sharp break in the curve so that the optimum storage is selected based on judgment. A "rule of thumb" for optimum storage would suggest that the capacity of the reservoir should be three to four times the median annual runoff. The optimum storage will vary depending on the demand pattern placed on the reservoir. Furthermore, the optimum storage for a firm annual
draft condition will be different from that for a draft with an allowable shortage condition. The optimum storage from a hydrologic perspective may be different than the optimum storage from an economic perspective.

b) Non-Storage Condition (i.e. Utilize Live Stream)

If withdrawal is from the live stream only, as would be the case for a spring backflood project, the water supply potential depends solely on the magnitude and variability of runoff from year to year. In this case, only a small withdrawal if any, can be expected on a firm basis (i.e. guaranteed annual diversion). Thus, a "reasonable" diversion can be expected only in a percentage of the years (i.e. not every year). The magnitude of withdrawal that is possible 70% of the time is considerably less than that which is possible 50% of the time. (The magnitude of the annual volume that is divertable 100% of the time would be zero for streams which may experience even one year of zero flow.)

2.4.2.2 Development of Water Supply Determination Procedures

The following information describes the background philosophy and investigative methods used in developing the procedures for estimating the project water supply potential. These procedures allow the estimation of the water supply potential of a basin for both storage and non-storage conditions. As the storage condition is the more common condition, a greater emphasis will be placed on this condition in the manual.

a) Storage Condition (i.e. Reservoir)

Ideally, the water supply potential of a storage project should be determined using a water supply model whose inputs would include a storage-area relationship, monthly inflows over a long period of time (50+ years), net evaporation, and specified demands (both magnitude and distribution). However, the basic data and computer model are not readily accessible to field offices. Thus, an approach that would be appropriate to field office facilities was developed.

The developed approach was one that would provide an estimate of the water supply potential of a project based solely on the following two parameters.

1. The median annual runoff into the reservoir.

2. A storage-draft curve appropriate to the geographic location of the reservoir.

The median annual runoff to the reservoir can be determined by using the median annual unit runoff map developed by the Hydrology Division for the three prairie provinces (Figure 2.4.7) and the effective drainage area tributary to the reservoir site as determined by the field office. A larger scale median annual unit runoff
map included in Hydrology Report #92 (see "References") makes it easier to estimate the median annual unit runoff.

Representative storage-draft curves were developed (see Figures 2.4.9 to 2.4.13) for five regions of the three prairie provinces (see Figure 2.4.8). The storage-draft relationships were developed for a hypothetical reservoir from a series of water budget simulations using the Hydrology Division's HY01 computer program. In order to develop "universal" storage-draft curves it was necessary to standardize the following components.

1. Elevation-Storage-Area Relationships

An actual reservoir was selected and accepted as being representative of a good prairie reservoir. The elevation-storage-area relationship of this "Standard" reservoir is as shown in Table 2.4.2.

This reservoir was used in all study simulations. The maximum capacity (300 dam$^3$) was adequate to store up to 4.75 times the median annual runoff for study simulations (see item 2 below for discussion of median annual runoff as a measure of inflow to the reservoir.)

In order to make the results of the water supply potential associated with the "Standard" reservoir applicable to all sizes of reservoirs in each of the five water supply regions, rates of "Annual Draft as a % of Median Annual Runoff" and "Reservoir Capacity as % of Median Annual Runoff" are utilized for the storage draft curves.

2. Inflow

Inflow to the study sites was based on recorded inflow (usually of larger streams) in the general area. The inflow from a larger stream was reduced by a constant factor so that the inflow to the reservoir had a median annual runoff of 57 dam$^3$. (It was necessary to make this adjustment to the inflow so that the inflow was in "harmony" with the size of the standard reservoir. Thus, a series of standardized water supply studies could be made.)

3. Reservoir Capacity

The reservoir capacity for each simulation was related to the median annual runoff at the study site. The standard reservoir whose capacities were 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0 and 4.75 times the median annual runoff were investigated.

4. Demand Patterns

Two uniform demand patterns were studied to determine their effect on reservoir draft: January to December and May to August.
5. Acceptability of Shortages

For each of the two demand patterns in 4 above, simulations were performed for the firm annual draft (no shortages) condition and for the condition of shortages in 30% of the years (i.e. demand is satisfied in 70% of the years).

6. Net Evaporation

Monthly net evaporation factors for study sites were based on the net evaporation as calculated by the PFRA Hydrology Division PFRA (see References).

b) Non-Storage Condition (i.e. Utilize Live Stream)

The inflows previously determined for the storage condition simulations were also used in the nonstorage condition analyses. A frequency analysis was performed on each inflow array to determine the runoff volumes that would be equalled or exceeded 50%, 60%, 70%, 80%, 90% and 100% of the time.

2.4.2.3 Procedures for Water Supply Potential Determination

The procedures for determining the water supply potential for both storage and non-storage conditions are described as follows.

a) Storage Condition (i.e. Reservoir)

This procedure can be used to determine either the available draft that can be obtained from a given storage or the storage required to provide a given demand. The following steps are common to both of these analyses.

1. Delineate the effective drainage area tributary to the damsite or study site on 1:50,000-scale NTS maps, making a field inspection as required.

2. Read the median annual unit runoff (MAUR) at the site location from Figure 2.4.7 (or from the more detailed map in Hydrology Report #92).

3. Calculate the median annual runoff (MAR) to the project or study site as follows:

   \[ MAR = MAUR \times \text{Effective Drainage Area} \]

4. Read the region number from Figure 2.4.8.

5. Select the storage-draft relationship that is appropriate to the site from regional runoff-draft curves in Figures 2.4.9 to 2.4.13. If the demand is given; compute the ratio of demand over median annual runoff. The storage of the project required to meet the given demand can be determined using the
selected storage-draft relationship. Similarly, if the project capacity is given, the magnitude of the draft that can be obtained from the project can be determined using the same selected storage-draft relationship. In either case, the user starts at the axis of the given component and uses the appropriate relationship to determine the required component.

b) Non-Storage Condition (i.e. Utilize Live Stream)

The following procedure is used to determine the water supply potential at a point of interest (e.g. diversion point).

1. Calculate the median annual runoff (MAR) at the diversion site and identify the region as in steps 1 to 4 in item a) above.

2. For a given exceedence (e.g. runoff volume exceeded 70% of the time), select the corresponding divertible runoff volume as a percent of MAR from Table 2.4.3.

3. Convert volume from a percent of MAR to dam³. Note that this procedure assumes that diversion facilities are adequate to utilize the available water.

2.4.2.4 Examples of Water Supply Potential Calculation

Examples for two types of projects (i.e. storage and non-storage) are provided to illustrate the use of the indicated procedures.

a) Storage Project

A water supply investigation utilizing a storage reservoir will be defined by one of three objectives:

1. to size the reservoir to obtain maximum draft; i.e. fully utilize the water supply potential of a basin. (The maximum reservoir size which utilizes the basin water supply potential is described as hydrological optimum storage);

2. to size the reservoir to meet a demand;

3. to determine the available draft for a given reservoir size.

Note that for a given watershed there are practical hydrological limits to reservoir size and available draft. Example of computations for each study objective are given below.

i) Size the Reservoir to Obtain the Maximum Draft

The following example illustrates the procedure required to estimate the water supply potential of a storage site located at 51° 00' 00" latitude and 105° 00' 00" longitude. The firm annual draft and draft with shortages in 30% of the years are required. The draft period is
May-August. The capacity of the reservoir is assumed to be at the hydrological optimum. There are no other users upstream or downstream of the storage site. The effective drainage area is 26.2 km².

The steps required to determine the water supply potential of the basin under the above conditions are listed as follows.

1. Using Figure 2.4.7, read the median annual unit runoff (MAUR) for the site to be 6.5 dam³/km².

2. Calculate the median annual runoff (MAR) as follows:
   
   \[
   \text{MAR} = \text{MAUR} \times \text{Effective Drainage Area} \\
   = 6.5 \times 26.2 \\
   = 170 \text{ dam}^3
   \]

3. From Figure 2.4.8 read Region No. 5 as the region applicable to the site location.

4. From Figure 2.4.13 (storage-draft curves for Region No. 5), subjectively determine the hydrological optimum storage. From the relevant curves, the optimum storage lies in the range of 300% to 400% of MAR. Arbitrarily select the optimum storage to be 350% of MAR. Reservoir storage is 3.50 x 170 or 595 dam³. From the appropriate curves (i.e. draft conditions), read the firm annual draft (55% of MAR) from curve B, and the draft with shortages in 30% of the years (111% of MAR) from curve D.

5. Convert the drafts from a percentage of MAR to dam³.

   - Firm Annual Draft = 55% of MAR
     = .55 x 170
     = 93.5 dam³

   - Draft with shortages in 30% of the years = 111% of MAR
     = 1.11 x 170
     = 187 dam³

ii) Size the Reservoir to Meet a Desired Demand

Assume that a firm annual draft of 63 dam³ (May-August draft period) is desired at the location given in section i) above. The required reservoir capacity to satisfy this demand is determined as follows.

1. Using Figure 2.4.7 read the median annual unit runoff (MAUR) for the site to be 6.5 dam³/Km².
2. Calculate the median annual runoff (MAR) as follows:

\[
\text{MAR} = \text{MAUR} \times \text{Effective Drainage Area} \\
= 6.5 \times 26.2 \\
= 170 \text{ dam}^3.
\]

3. From figure 2.4.8 read Region No. 5 as the region applicable to the site location.

4. Convert desired draft (63 dam\(^3\)) into a percentage of MAR (i.e. \((63/170) \times 100 = 37\%\)).

5. From Figure 2.4.13, select relevant storage-draft curve (i.e. curve B).

6. For a firm draft of 37\% of MAR, read a reservoir capacity of 157\% of MAR from curve B.

7. Convert reservoir capacity from a percentage of MAR to dam\(^3\).

\[
\text{Reservoir Capacity} = 157\% \text{ of MAR} \\
= 1.57 \times 170 \\
= 267 \text{ dam}^3
\]

iii) For a Given Reservoir Size, Determine the Draft

Assume that reservoir drafts are required for a given reservoir storage (capacity of 680 dam\(^3\)) at the location and with the relevant draft conditions described in Section (i) above.

1. Proceed as per steps 1, 2 and 3 in Section (i) above to obtain MAR (170 dam\(^3\)) and region number (No. 5) applicable to the site.

2. Express the reservoir capacity (680 dam\(^3\)) as a percent of MAR (170 dam\(^3\)) as follows

\[
\frac{680}{170} \times 100 = 400\%
\]

3. From Figure 2.4.13 (Storage-Draft Curves for Region No. 5) read the appropriate drafts as follows: For reservoir capacity equal to 400\% of MAR read the firm annual draft (57\% of MAR) from curve B, and the draft with shortages in 30\% of the years (113\% of MAR) from curve D.

4. Convert the draft from a percentage of MAR to a draft volume (dam\(^3\))

\[
\text{Firm Annual Draft} = 57\% \text{ of MAR} \\
= 0.57 \times 170 \\
= 96.9 \text{ dam}^3
\]
Draft with shortages in 30% of the year = 113% of MAR
= 1.13 \times 170
= 192 \text{ dam}^3

b) Diversion Project (i.e. no storage)

An assessment of the backflood potential is required at the same location as the previous example (a). Assume that diversion is from a live stream and the capacity of the diversion facilities (structure and canal) will not limit the desired divertible flow. For practical purposes, it is assumed that the diversion facilities would be designed to allow the desired diversion in no more than 50% of the years (i.e. the diversion facilities would not normally be so large as to divert all of the flow in the rare high-flow years).

The step-by-step procedure required to determine the water supply potential of a basin by diverting from a live stream is illustrated by the following example.

1. Proceed as per Steps 1, 2 and 3 in section i) above to obtain MAR (170 \text{ dam}^3) and region number (i.e. No. 5) applicable to the site.

2. From Table 2.4.3, read divertible flow factors applicable to Region No. 5. For example, the volume of water available for diversion 70% of the time (i.e. in 70% of the years) is 52% of MAR.

3. Convert the volume of water that would be available in 70% of the years from a percentage of MAR to \text{dam}^3:
\[0.52 \times 170 = 88.4 \text{ dam}^3\]

2.4.2.5 Uses and Limitations

The recommended approach for assessing the water supply potential applies to small watersheds (50 km² or less) that have not been appreciably altered by upstream development. To estimate the uncommitted (i.e. available) water in a basin that has upstream users, calculate the "present use" median annual runoff by deducting allocated upstream uses and average annual reservoir evaporation losses from the natural median annual runoff derived from the MAUR map. Provincial water rights licensing agencies allocate uses and average reservoir evaporation losses for projects.

The relationships that have been developed for this manual are very general and do not reflect conditions that may be experienced in unique basins. Complicated systems or apparently unique situations should be referred to a qualified professional engineer for assessment.

Small prairie reservoirs vary in their efficiency to carry over storage from one year to the next. The smaller the water surface for a given storage, the less the evaporation and thus the higher the reservoir efficiency. This manual was developed for a reservoir that a water resource person would call a "good" prairie reservoir and thus is directly applicable to reservoirs that are in
the "good" category. The water supply potential for a "poor" reservoir (e.g. shallow, dish-shaped reservoir) or for a "extra good" reservoir (e.g. standard PFRA dugout) cannot be estimated directly from this manual without some subjective judgement. However, the manual can be used to obtain a perspective on the water supply potential of reservoirs that are less or more efficient than the "good" reservoir used in developing this manual.

The term "storage" as it applies to water supply calculations, is assumed to be "live storage" (i.e. usable storage). Unavailable storage (i.e. dead storage) must be subtracted from reservoir capacity before making water supply calculations using the procedures recommended in this manual.

2.4.3 Hydrology Summary Report

The hydrological investigations performed by the project designer are summarized in a standard report "Hydrology Summary Report". The report briefly describes the watershed area, lists the various parameters used in the hydrology studies and specified the water supply potential and flood potential.

This report provides the basis for selecting the reservoir Full Supply Level (FSL) and sizing the spillway system and is considered a necessary input for the project design.
Table 2.4.1. Drainability Factors Selection Chart

<table>
<thead>
<tr>
<th>FACTOR</th>
<th>RANGE OF PARAMETERS</th>
<th>DF VALUES</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basin Slope (DF1)</td>
<td>&lt; 0.0005 m/m</td>
<td>0.0</td>
</tr>
<tr>
<td></td>
<td>0.001 m/m</td>
<td>0.1</td>
</tr>
<tr>
<td></td>
<td>0.002 m/m</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td>0.003 m/m</td>
<td>0.3</td>
</tr>
<tr>
<td></td>
<td>&gt; 0.004 m/m</td>
<td>0.4</td>
</tr>
<tr>
<td>Drainage Channel Development (DF2)</td>
<td>no channels, or a few channels and many sloughs</td>
<td>0.0</td>
</tr>
<tr>
<td></td>
<td>moderate amount of channels with a few sloughs</td>
<td>0.15</td>
</tr>
<tr>
<td></td>
<td>extensive channel development with essentially no significant depressional storage (sloughs)</td>
<td>0.3</td>
</tr>
<tr>
<td>Basin Shape (DF3)</td>
<td>long and thin (length to width ratio &gt; 10)</td>
<td>0.0</td>
</tr>
<tr>
<td></td>
<td>moderately compact</td>
<td>0.1</td>
</tr>
<tr>
<td></td>
<td>very compacted (length to width ratio &lt; 2)</td>
<td>0.2</td>
</tr>
<tr>
<td>Vegetative Cover (DF4)</td>
<td>heavy cover of trees and bush</td>
<td>0.0</td>
</tr>
<tr>
<td></td>
<td>cropped land and/or good pasture</td>
<td>0.05</td>
</tr>
<tr>
<td></td>
<td>poor or very little vegetative cover</td>
<td>0.10</td>
</tr>
<tr>
<td>Soil Texture (DF5)</td>
<td>sand</td>
<td>0.0</td>
</tr>
<tr>
<td></td>
<td>loam</td>
<td>0.05</td>
</tr>
<tr>
<td></td>
<td>clay</td>
<td>0.10</td>
</tr>
</tbody>
</table>

Notes:  
1. Select the DF value for each factor, interpolating between tabulated values if required.
2. The overall drainability factor for the study basin is the sum of the five individual DF values.
### Table 2.4.2: Elevation-Storage-Area Relationship for Standard Reservoir

<table>
<thead>
<tr>
<th>Elevation (metres)</th>
<th>Storage (dam³)</th>
<th>Area (hectares)</th>
</tr>
</thead>
<tbody>
<tr>
<td>97.210</td>
<td>0.0</td>
<td>0.00</td>
</tr>
<tr>
<td>99.000</td>
<td>1.0</td>
<td>0.05</td>
</tr>
<tr>
<td>100.000</td>
<td>4.0</td>
<td>0.38</td>
</tr>
<tr>
<td>100.500</td>
<td>6.0</td>
<td>0.62</td>
</tr>
<tr>
<td>101.000</td>
<td>11.0</td>
<td>0.93</td>
</tr>
<tr>
<td>101.500</td>
<td>17.0</td>
<td>1.26</td>
</tr>
<tr>
<td>102.000</td>
<td>26.0</td>
<td>1.66</td>
</tr>
<tr>
<td>102.500</td>
<td>36.0</td>
<td>2.12</td>
</tr>
<tr>
<td>103.000</td>
<td>48.0</td>
<td>2.56</td>
</tr>
<tr>
<td>103.500</td>
<td>64.0</td>
<td>3.10</td>
</tr>
<tr>
<td>104.000</td>
<td>80.0</td>
<td>3.62</td>
</tr>
<tr>
<td>104.500</td>
<td>100.0</td>
<td>4.23</td>
</tr>
<tr>
<td>105.000</td>
<td>122.0</td>
<td>4.83</td>
</tr>
<tr>
<td>105.500</td>
<td>149.0</td>
<td>5.52</td>
</tr>
<tr>
<td>106.000</td>
<td>177.0</td>
<td>6.54</td>
</tr>
<tr>
<td>106.500</td>
<td>214.0</td>
<td>8.10</td>
</tr>
<tr>
<td>107.000</td>
<td>254.0</td>
<td>9.24</td>
</tr>
<tr>
<td>107.500</td>
<td>300.0</td>
<td>10.25</td>
</tr>
</tbody>
</table>

### Table 2.4.3: Table of Divertible Flow Factors

<table>
<thead>
<tr>
<th>Region No.</th>
<th>Annual Volumes Divertible for the Indicated Percentage of the Time as a Percent of the Median Annual Runoff</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>50%</td>
</tr>
<tr>
<td>1</td>
<td>100</td>
</tr>
<tr>
<td>2</td>
<td>100</td>
</tr>
<tr>
<td>3</td>
<td>100</td>
</tr>
<tr>
<td>4</td>
<td>100</td>
</tr>
<tr>
<td>5</td>
<td>100</td>
</tr>
</tbody>
</table>

Example: For Region No. 5, the minimum volume of flow that can be diverted 70% of the time (i.e. in 70% of the years) is 52% of the median annual runoff.
Example: For a moderately well-drained basin of 20 km², the Index Flood is 1.75 m³/s.

Note: The curve values represent the drainability of a basin, taking into account five drainability factors: basin slope, drainage development, basin shape, vegetative cover and soil texture.

Fig. 2.4.1 Index Flood Selection Graph
Example: For a 20 km² drainage basin, the multipliers of the Index Flood used to calculate the 1:10, 1:20, 1:50, and 1:100 flood peaks are 4.5, 7.1, 11.5 and 16.1, respectively.

Fig. 2.4.2 Index Flood Multiplier Selection Graph
Fig. 2.4.3 Rainfall Isopleths for a 1:100 Six-hour Rain
Example: For a study site having a 1:100 six-hour rainstorm of 57 mm (see Figure 2.4.3), the calculated unadjusted flood peak should be multiplied by an adjustment factor of 0.811.

Fig. 2.4.4 Flood Peak Adjustment Factor Based on the Rainfall Component
Example: For a study site having a May-June net evaporation of 89 mm (see Figure 2.4.5), the calculated unadjusted flood peak should be multiplied by an adjustment factor of 0.863.

Fig. 2.4.6 Flood Peak Adjustment Factor Based on the Soil Moisture Condition Using the Average May--June Net Evaporation as an Indicator
Fig. 2.4.7 Median Annual Unit Runoff for the Prairie Provinces

Notes: 
1. Isopleths of median annual unit runoff are in dam³/km².
2. For greater ease in determining median annual unit runoff, use the larger scale Median Annual Unit Runoff map included in Hydrology Report #92, October 1978.
Fig. 2.4.8 Runoff—Draft Regions for the Prairie Provinces
LEGEND:

A Firm annual draft for January-December draft period.
B Firm annual draft for May-August draft period.
C Draft with shortages in 30% of the years for January-December draft period.
D Draft with shortages in 30% of the years for May-August draft period.

Notes: 1. See Figure 2.4.8 for identification of regions.
2. Drafts are for a uniform demand (i.e. constant from month to month).

Fig. 2.4.9 Storage-Draft Curves for Region No. 1
LEGEND:

A Firm annual draft for January–December draft period.
B Firm annual draft for May–August draft period.
C Draft with shortages in 30% of the years for January–December draft period.
D Draft with shortages in 30% of the years for May–August draft period.

Notes: 1. See Figure 2.4.8 for identification of regions.
2. Drafts are for a uniform demand (i.e. constant from month to month).

Fig. 2.4.10 Storage–Draft Curves for Region No. 2
LEGEND:

A Firm annual draft for January-December draft period.
B Firm annual draft for May-August draft period.

C Draft with shortages in 30% of the years for January-December draft period.
D Draft with shortages in 30% of the years for May-August draft period.

Notes: 1. See Figure 2.4.8 for identification of regions.
2. Drafts are for a uniform demand (i.e. constant from month to month).

Fig. 2.4.11 Storage-Draft Curves for Region No. 3
Fig. 2.4.12 Storage—Draft Curves for Region No. 4

Legend:

A Firm annual draft for January—December draft period.
B Firm annual draft for May—August draft period.
C Draft with shortages in 30% of the years for January—December draft period.
D Draft with shortages in 30% of the years for May—August draft period.

Notes: 1. See Figure 2.4.8 for identification of regions.
2. Drafts are for a uniform demand (i.e. constant from month to month).
Figure 2.4.13 Storage-Draft Curves for Region No. 5

Legend:
A Firm annual draft for January-December draft period.
B Firm annual draft for May-August draft period.
C Draft with shortages in 30% of the years for January-December draft period.
D Draft with shortages in 30% of the years for May-August draft period.

Notes:
1. See Figure 2.4.8 for identification of regions.
2. Drafts are for a uniform demand (i.e. constant from month to month).
## HYDROLOGY SUMMARY REPORT

**Project Title / Owner:**

**Location:**

**Designer:**

**Date:**

### General Site Characteristics

<table>
<thead>
<tr>
<th>Effective Drainage Area:</th>
<th>km²</th>
<th>Gross Drainage Area:</th>
<th>km²</th>
</tr>
</thead>
</table>

**Watershed Description:**

**Schematic Watershed Map**

### Water Supply Potential

**Runoff-Draft Region:**

- Median Annual Unit Runoff: ________ dam³/km²;

**Median Annual Runoff:** ________ dam³

**Project Demand:**

a) **Live Stream Diversion (No Reservoir):**

- Divertable Annual Volume: 50% of the time ________ dam³; 70% of the time ________ dam³

b) **Storage Condition:**

- Reservoir capacity as a % of Median Annual Runoff: ________; Reservoir Capacity: ________ dam³

**Firm Annual Draft (0% shortages):** (Jan - Dec. Demand) ________ dam³; (May - Aug. Demand) ________ dam³.

**Annual Draft with shortages in 30% of the years:** (Jan - Dec. Demand) ________ dam³; (May - Aug. Demand) ________ dam³.

### Flood Potential

**Overall Drainability Factor:** ________

**Index Flood (1 : 2):** ________ m³/s

**Index Flood Multiplier:**

- (1 : 10) ________; (1 : 50) ________; (1 : 100) ________

**Rainfall Adjustment Factor:**

- (1 : 10) ________; (1 : 50) ________; (1 : 100) ________

**Soil Moisture Adjustment Factor:**

- (1 : 10) ________; (1 : 50) ________; (1 : 100) ________

**Instantaneous Flood Peaks (m³/s):**

- (1 : 10) ________; (1 : 50) ________; (1 : 100) ________

---

*Figure 2.4.14*
2.5 GEOTECHNOLOGY

The Geotechnical Investigation Section describes an engineering soil classification system, field methods, and criteria for classifying foundation conditions and embankment materials for construction of small earth dams. Upon completion of identifying the foundation and embankment materials, the project designer assesses the foundation and embankment materials as "Good, Fair or Poor" with respect to their adequacy for construction of a dam facility. The assessment is documented in the form of a "Geotechnical Assessment Report". If the project designer is unable to make an assessment on the nature of the foundation or borrow materials due to the conditions or circumstances at a particular site, he shall consult with a qualified professional engineer for advice and/or performance of a geotechnical assessment.

2.5.1 Soil Classification

A classification system developed by Dr. A. Casagrande for the purpose of classifying and identifying soils for airfield construction was modified by the Corps of Engineers and Bureau of Reclamation into the Unified Soil Classification System (USCS) so that it applied to all types of soils engineering. PFRA utilizes a slightly modified Unified Soil Classification System.

The USCS is used to identify soil types according to their textural and plasticity qualities and to group soils with respect to their behavior as an engineering material. The classification system groups soil types into major divisions and subdivisions based on laboratory tests and visual examination. This section describes the basic classification tests and criteria used to identify and differentiate soil types. Details of test methods are described in volume 04.08 of American Society for Testing and Materials (ASTM) Standards.

2.5.1.1 Laboratory Tests

The following tests are used for identification of basic soil properties and soil groups.

1) Water Content - Water content is the ratio of the mass of water to the mass of soil solids, usually expressed as a percentage. The mass of a moist sample is determined before and after drying the sample in an oven. The water content is the ratio of the difference between wet and dry mass, to the dry mass.

2) Density - Density is mass per unit volume. In the laboratory, the volume of a sample is usually determined by immersion in water with the volume of water displaced being the volume of the sample. In the field, the volume of a hole from which a measured mass of soil is removed is determined by filling the hole with a measured volume of another substance as described in Section 4.4. The soil mass can be determined in a wet or
dry condition which, when divided by the volume, describes the wet or dry density. Density may be determined in a natural (in situ) state or in a remoulded state such as a compacted soil.

3) **Grain-Size Distribution** - Grain-size distribution of coarse-grained samples is determined by shaking dried gravel size and smaller material through a series of progressively finer standard sieves. Boulder and cobble-sized material is physically removed and measured with a scale or ruler. All lumps must be broken down into individual grains. The ratio of the dry mass retained on each sieve to the total dry mass of the sample, expressed as a percentage, shows a gradation curve when plotted on semi-log graph paper. Figure 2.5.1 is an example of grain-size distribution.

Grain-size distribution of fine-grained soils (silt and clay) are not as important as plasticity characteristics which govern behavior; however, it should be known that silt sizes are larger than clay sizes and exhibit relatively low plasticity.

4) **Atterberg Limits** - Clay soils exhibit plasticity characteristics in the presence of water which can be measured by standard tests. Three main states of soil consistency, dependent on the amount of water in the soil, are recognized; (1) Liquid state in which the soil behaves like a viscous fluid, (2) Plastic state in which the soil can be rapidly deformed without cracking or crumbling and (3) Solid or semi-solid state in which the soil will crack when deformed. Atterberg defined the following limits between these states; between (1) and (2) the liquid limit and between (2) and (3) the plastic limit. These limits are expressed as percent water content based on the dry mass of the soil passing the 425 micrometre (No. 40) sieve. The magnitude of the natural water content of a soil in relation to its liquid and plastic limits is indicative of the engineering characteristics of the soil in its natural state.

Liquid limit apparatus is used to determine the water content of a sample at its liquid limit. The plastic limit is the water content at which the soil begins to crumble into 6 to 8 mm long segments when rolled out into threads 3 mm in diameter. The numerical difference between the liquid and plastic limit is known as the plasticity index which is the range of water content within which a soil exhibits plasticity. The liquid limit, plastic limit and plasticity index are often expressed as a series of three numbers such as 46-14-32.

2.5.1.2 **Identification of Soil Groups**

Soils can be classified into three major divisions by means of visual examination and simple field tests as described
herein. Classification into sub divisions can also be made by visual examination with some degree of success. In borderline cases it may be necessary to verify the classification by laboratory tests. Field test methods are generally adequate for preliminary classification and may be used to advantage in grouping soils so that a minimum number of laboratory tests need be performed.

The three major divisions of the Unified Soil Classification System utilized by PFRA are:

1) Coarse-grained soils - More than 50% by mass larger than the 75 um (No. 200) sieve applicable to materials passing the 75 mm sieve.

2) Fine-grained soils - More than 50% by mass smaller than the 75 um (No. 200) sieve applicable to materials passing the 75 mm sieve.

3) Highly organic soils - Peat, muskeg and swamp materials.

The names "boulders", "cobbles", "gravel", "sand" and "fines (silt or clay)" are used to designate the size ranges of soil particles. PFRA differentiates between cobbles and boulders at a diameter of 200 mm whereas ASTM uses 300 mm. Gravel and sand size ranges are subdivided further. Limiting boundaries between the various size ranges have been established at certain sieve sizes in accordance with the following tabulation:

<table>
<thead>
<tr>
<th>Components</th>
<th>Size Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Boulders</td>
<td>Greater than 200 mm</td>
</tr>
<tr>
<td>Cobble</td>
<td>200 mm to 0.075 mm</td>
</tr>
<tr>
<td>Gravel</td>
<td>75 mm to 4.75 mm (No. 4)</td>
</tr>
<tr>
<td>Coarse Gravel</td>
<td>75 mm to 19 mm</td>
</tr>
<tr>
<td>Fine Gravel</td>
<td>19 mm to 4.75 mm (No. 4)</td>
</tr>
<tr>
<td>Sand</td>
<td>4.75 mm (No. 4) to 0.075 mm (No. 200)</td>
</tr>
<tr>
<td>Coarse Sand</td>
<td>4.75 mm (No. 4) to 2.00 mm (No. 10)</td>
</tr>
<tr>
<td>Medium Sand</td>
<td>2.00 mm (No. 10) to 0.425 mm (No. 40)</td>
</tr>
<tr>
<td>Fine Sand</td>
<td>4.25 mm (No. 40) to 0.075 mm (No. 200)</td>
</tr>
<tr>
<td>Fines (silt or clay)</td>
<td>Below 0.075 mm (No. 200)</td>
</tr>
</tbody>
</table>

Soils seldom exist in nature separately as sand, gravel, or any other single component, but are usually found as mixtures with varying proportions of particles of different sizes. Each component part contributes its characteristics to the soil mixture. In the Unified Soil Classification System the soil is given a descriptive name and a letter symbol indicating its principle characteristics. The following description of detailed soil classification and identification criteria and procedures is summarized on Tables 2.5.1 and 2.5.2 for handy reference.
Coarse-grained soils are subdivided into gravel and gravelly soils (symbol G), and sands and sandy soils (symbol S). In general practice there is no clear-cut boundary between gravelly soils and sandy soils and, as far as behavior is concerned, the exact point of division is relatively unimportant. For purposes of identification, coarse-grained soils are classed as gravels (G) if more than 50% of the coarse fraction (retained on 0.075 mm (No. 200) sieve) is larger than the 4.75 mm (No. 4) sieve and as sands (S) if 50% or more of the coarse fraction passes the 4.75 mm (No. 4) sieve. The gravel (G) and sand (S) groups are each divided into four secondary groups as follows:

a) Well-graded material with less than 5% fines. Use symbol W forming groups GW and SW. The grain-size distributions of well-graded materials generally plot as smooth and regular concave curves with no sizes lacking and no excess of material in any size range.

b) Poorly-graded material with less than 5% fines. Use symbol P forming groups GP and SP. The grain size distributions of poorly graded materials may be near vertical indicating a uniform size, straight line gradations indicating an excess of material in a size range or humped or bowed indicating a size range is lacking.

c) Coarse material with greater than 12% nonplastic or low plastic fines. Use symbol M forming groups GM and SM. Due to the fines content, the gradation of these materials is not considered significant so both well and poorly-graded materials are included.

d) Coarse material with greater than 12% plastic fines. Use symbol C forming groups GC and SC. Due to the fines content, the gradation of these materials is not considered significant so both well and poorly graded materials are included.

These four subdivisions are differentiated due to the different behavior of well and poorly graded materials and coarse materials with high plastic or nonplastic fines content.

Borderline soils with a fines content between 5 and 12% are assigned a double symbol depending on both the gradation characteristics of the coarse fraction and the plasticity characteristics of the fines fraction. Borderline classifications include GWGM, GWGC, GPGM, GPGC, SWSM, SWSC, SPSM and SPSC.
Coarse-grained soils must meet both the following gradation criteria to be well graded:

**Uniformity Coefficient**

\[ Cu = \frac{D_{60}}{D_{10}} \]

greater than 4 (gravel)

\[ D_{10} \]

greater than 6 (sand)

**Coefficient of Curvature**

\[ C_c = \frac{(D_{30})^2}{D_{60} \times D_{10}} \]

between 1 and 3

where

- \( D_{60} \) = grain diameter at 60 percent passing
- \( D_{30} \) = grain diameter at 30 percent passing
- \( D_{10} \) = grain diameter at 10 percent passing

These criteria ensure the material is not uniform and has no excess or lack of particles in any size range. The grain size distribution should show a curvature typical of well graded materials as shown on Figure 2.5.1.

Field classification of coarse-grained soils requires some experience to determine gradation characteristics and the amount and type of fines present. Gradation characteristics to be visually examined include gravel content and the presence of equal amounts by mass of all sizes within the range of sizes present in the sample. The amount of fines present can be estimated by visual examination of the sample or by adding water and observing how dirty the water becomes. The plasticity of the fines can be estimated by methods described in the following section.

2) **Fine-Grained Soils**

Fine-grained soils are subdivided on the basis of liquid limit, namely, low (symbol L), medium (symbol I) or high (symbol H). The terms "silt" and "clay" are used respectively to distinguish soils exhibiting lower plasticity from those with higher plasticity. The "A" line on the plasticity chart Table 2.5.1 divides clay soils above the line from silt soils below the line on the basis of liquid limit and plasticity index. Divisions between low, medium and high plasticity occur at liquid limits of 30 and 50. Medium plasticity is a subdivision defined by PFRA and is assigned to clays only. Further subdivisions are as follows:

a) Inorganic silt. Use symbol M forming groups ML and MH. The presence of clay in the silt will cause plasticity of the material.
b) Inorganic clay. Use symbol C forming groups CL, CI, and CH.

c) Organic silt and clay. Use symbol O forming groups OL and OH. These soils have a plasticity range that corresponds with the ML and MH groups, respectively.

Field classification of fine-grained soils involves testing for dilatancy (reaction to shaking), examination of plasticity characteristics and determination of dry strength. Observations of colour and odor are of value, particularly for organic soils.

Testing of dilatancy is performed on a moist sample that can be easily held in one hand. The sample is alternately shaken horizontally in the open cupped palm of one hand, by striking vigorously against the other hand several times, and then squeezed between the fingers. A fine-grained soil that is nonplastic or exhibits very low plasticity will become "livery" and show free water on the surface while being shaken. Squeezing will cause the water to disappear from the surface and the sample to stiffen and finally crumble under increasing finger pressure, like a brittle material. Clay soils show no reaction to this test.

Examination of the plasticity characteristics of fine-grained soils or of the fine fraction of coarse-grained soils is made with a small moist sample of material moulded to the consistency of putty. If the soil is too dry, water must be added and if it is sticky, the specimen should be spread out in a thin layer and allowed to lose some moisture by evaporation or by reworking. The sample is rolled by hand on a smooth surface or between the palms into a thread about 3 mm in diameter. The thread is then folded and rerolled repeatedly. During this manipulation the moisture content is gradually reduced and the specimen stiffens, finally loses its plasticity, and crumbles when the plastic limit is reached.

Toughness is the resistance to deformation of the soil at the plastic limit moisture content. As the specimen is rolled into threads and formed into a lump for rerolling, toughness can be described in terms of low, medium or high. The toughness is low when only slight pressure is required to roll the thread to near the plastic limit and the thread and the lump are weak and soft. The toughness is medium when medium pressure is required to roll the thread to near the plastic limit and the thread and the lump have medium stiffness. The toughness is high when considerable pressure is required to roll the thread to near the plastic limit and the thread and the lump have very high stiffness. ML soil have no toughness, OL soils have low toughness and MH and OH have low to medium toughness. Soils containing organic material, especially those with a low plasticity index, form threads that are very
soft and spongy near the plastic limit. Inorganic clays have increasing toughness with increasing plasticity.

Dry strength is the resistance of a piece of dried soil to crushing which is an indication of the character of fine-grained soils. A sample with coarse particles removed is molded to the consistency of putty with addition of water if necessary. The moist soil is allowed to dry (in oven, sun or air) and is then crumbled between the fingers. Soils with slight dry strength crumble readily with very little finger pressure. All nonplastic ML and MH soils have almost no dry strength. Organic silts and lean organic clays of low plasticity (OL), as well as very fine sandy soils (SM), have slight dry strength. Soils of medium dry strength require considerable finger pressure to powder the sample. Most clays of the CL group and some OH soils exhibit medium dry strength. This is also true of the fine fraction of gravelly and sandy soils having a clay binder (GC and SC). Soils with high dry strength cannot be broken by finger pressure; however, the specimen will break into pieces between the thumb and a hard surface. Soil specimens with very high dry strength cannot be broken between the thumb and a hard surface. High to very high dry strength is indicative of most CH clays, as well as some organic clays of the OH group having very high liquid limits and located near the "A" line.

Colour is often helpful in distinguishing between different soil strata and may be useful for identifying individual soils. The colour of moist soil should be used for identification as soil colour may change markedly on drying. Dark shades of grey or brown, including almost black colours, are indicative of fine-grained soil containing organic matter (OL, OH). In contrast, brighter colours, including medium and light grey, olive green, brown, red, yellow, and white are generally associated with inorganic soils.

The distinctive odor of organic soils of the OL and OH groups can be used as an aid in the identification of such materials. This odor is especially apparent from fresh samples. It gradually diminishes on exposure to air, but can be revived by heating a wet sample.

3) Highly Organic Soils

The field identification of highly organic soil (group Pt) is relatively easy inasmuch as these soils are characterized by undecayed or partially carbonized particles of leaves, sticks, grass, and other vegetable matter which impart to the soil a typical fibrous texture. The colour ranges generally from various shades of dull brown to black. A distinct organic odor is also characteristic of the soil. The water content is usually very high. Another aid in identification of these soils may be the location of the soil with respect
to topography as low-lying, swampy areas usually contain highly organic soils.

2.5.2 Field Investigations

There are usually two types of field investigation performed when assessing a potential dam site. A reconnaissance inspection serves as an introduction to the potential dam site, provides an insight and appreciation of any site features or constraints, and aids in planning future subsurface investigations. A subsurface investigation provides the means for identifying foundation stratigraphy and conditions, obtaining soil samples, and assessing the geotechnical suitability of a particular location. In a situation where the description of the foundation and materials cannot be ascertained with a sufficient degree of confidence, a qualified professional engineer should be consulted for assistance or advice.

General information regarding the geotechnical characteristics of a site can be obtained from a review of the available literature which includes:

- Geological maps and reports
- Pedology (soil survey and land use) maps
- Case histories in the area
- Topographic contour maps
- Water well drillers' records
- Forestry and land use maps.

The complexity of soil deposits may cause difficulties in correctly assessing the geotechnical characteristics of a site. Prior knowledge gained from the sources listed above will reduce the possibility of misjudgments during field investigations.

Investigations to obtain geological information for identifying potential geological hazards and preparing the "geological damsiten description" as per Section 2.3 "Geology" are conducted before hand or concurrently with the geotechnical investigations.

2.5.2.1 Reconnaissance Investigation

There is a relationship between topographic features or landforms and the characteristics of subsurface soils. The ability to recognize terrain features during a field reconnaissance in combination with a basic understanding of geological processes can be of great assistance in making a general assessment of foundation conditions and construction materials. This section describes some type of features which reflect material characteristics and potential problem areas to assist in developing a concept of the physical features of the geologic units in the landscape and the materials of which they are
composed. For information on the classification and engineering characteristics of geological materials and common relationships among geological units refer to Section 2.3 "Geology". Experience at observation and deduction will improve accuracy and reliability of assessment based on a field reconnaissance.

Soil Exposures

Soil types can be identified by examining exposures in dugouts, highway cuts or slope failures. Surfacial dried soils should be scraped off to expose the soil at its natural water content which shows an in situ colour representative of the soil in its natural state. Animal burrow excavations can also provide information on surficial soil type.

Erosion Features

Surfacial water run-off produces water erosion features such as gullies that often reflect the textural characteristics of the exposed materials. Short, steep, V-shaped gullies are associated with granular materials which have a high infiltration rate while long gullies with rounded cross-section are associated with fine-grained plastic soils which have a low infiltration rate. Uniform materials between the two extremes of clay size and gravel-size will have slopes and cross-sections commensurate with their grain size distribution. Clay till may have a cross-sections similar to sand but slightly more rounded whereas the gully length may be similar to silty clay and much longer than sand or gravel. The presence of a water table in coarse-grained soils may cause a gully to have a flat bottom.

The significance of gullies as an indicator of soil texture is reduced somewhat by climatic influence; however, changes in the gradient or cross-section of a gully may indicate a change in soil type or the influence of groundwater.

Unstable Slopes and Groundwater

Water is a major factor in slope instability because water pressure reduces the frictional resistance of soil particles to displacement. Many slope failures occur as a result of an increase in groundwater level caused by events such as excessive rainfall, removal of vegetation by cutting or fire, urbanization as well as other activities of man such as irrigation or reservoir impoundment against a dam. Natural slopes can fail if they are oversteepened due to streamflow erosion at the toe of the slope.

Slope failures occur in a variety of configurations, depending on geological boundaries and nature of the sediments. Failures often take place along weak zones such as bentonite layers. Typically, the upper portion of a slope failure consists of a steep surface exposing undisturbed ground called a scarp which is part of the failure surface. The main body of the slope failure is often like a spoon or shovel-like scoop pushed downslope in a circular arc.
type of failure. In plan view the shape is like an amphitheatre. Sometimes a large slope failure comprises a series of retrogressive smaller failures with each successive small failure occurring further upslope. The debris pile below the main scarp generally consists of a series of sub-parallel curved ridges or down-faulted blocks. Failed slopes generally have a hummocky or hilly appearance which has disturbed natural drainage channels and may have trapped small ponds on high ground which would normally be well drained.

Many slopes have failed in the distant past so the surface ridges and scarps have become obscured by erosion and vegetation. These dormant slope failures are easily reactivated and are therefore important to recognize; however, they are often difficult to detect by untrained observers. Clues to the existence of old slope failures include curved scarps or ridges, hummocky slopes or debris piles as well as unusual drainage patterns.

Seepage areas are often associated with slope instability and can usually be detected by concentrations of relatively lush vegetation. Salinity indicates a groundwater discharge area which might be associated with artesian water pressures or local or regional groundwater flow patterns. Piping or sub-surface erosion of soil particles by groundwater forms small erosion gullies where the soil particles have been carried away or sand boils where eroded soil particles are deposited in a ring around the erosion hole. The location and elevation of seepage areas or springs should be determined and flow rates estimated whenever possible.

**Vegetation Indicators**

Natural vegetation can be a useful indicator of terrain conditions including moisture content, groundwater conditions and stratigraphic changes in soil types. In a generally dry environment like the prairies, the competition for survival of vegetation is intense, resulting in clearly defined plant types which can be interpreted to define subsurface conditions with consideration of environmental factors. These factors include shading from the sun, wind forces causing direct damage or drying of the soil as well as topographic control of runoff and infiltration.

Trees are less sensitive to annual climatic variations than annual plants so their presence is a more reliable indicator of subsurface conditions; however, portions of the southern prairies may be naturally devoid of trees. In addition, some types of vegetation grow under a wide variety of conditions and are therefore poor indicators of terrain conditions. Although very few people possess detailed knowledge of vegetation types and their characteristic habitat, there is always something which can be interpreted from the existing vegetation, even the total lack of it. The following general descriptions of vegetation indicators provide some examples to assist in defining subsurface conditions in a reconnaissance investigation. Due to the complexity of factors affecting vegetation growth, interpreted subsoil conditions should be confirmed by other means.
Groundwater discharge areas such as sloughs and springs usually contain slough grass, sedge, bulrushes and cattails. Where these discharge areas are saline, salt-tolerant plant species called halophytes exist. One such species common to the prairies is samphire which has a characteristic reddish colour. Other halophytes include kochia and wild barley.

Flooding may occur in meadows surrounding sloughs. Tolerant species to flooding are willow and black poplar; however, these species are not salt tolerant and will be absent if salinity is present. Native trees which require moist soil conditions include Saskatoon berry, chokecherry, dogwood, cranberry, birch, spruce, fir, pine and aspen. Birch, spruce, fir, pine and aspen are generally not tolerant to flooding. It should be noted that there are usually more than one variety of each species and each variety has its own unique characteristics and optimum habitat.

Springs generally occur in pervious materials at a stratigraphic boundary which may divide massive units of different materials or through a pervious layer within a particular soil type. Willow and black poplar are known to thrive around such areas with slough grass, sedge, bulrushes and cattails in permanently flooded locations. Spruce, aspen and birch grow well in moist clay till while pine and fir like sandy soils and sandstones if sufficient moisture is available. Clay shale usually has weakly developed vegetation due to lack of nutrients. As clay shale and sandstone are often interbedded there may be denser vegetation along sandstone outcrops if water is present. Sands and gravels with good drainage may grow pine or aspen if precipitation is adequate. Coarse-grained soils with no water table support only hardy prairie grasses.

Tilted and overturned trees can be indicative of recent slope instability. Thistles are a non-native species which are indicators of recently disturbed ground. Curved tree trunks are indicative of past slope movement.

These vegetation indicators along with other observations made during the reconnaissance investigation will help determine the type and extent of the soils occurring at a site as well as moisture and groundwater conditions. This information will assist in defining the limits of a subsurface investigation program.

2.5.2.2 Subsurface Investigation

A subsurface investigation to confirm visual observations is usually performed on one or two sites after assessing the results of the reconnaissance investigation and other studies on a number of alternative sites. Auger holes or backhoe test pits provide a practical economical method of obtaining subsurface information. The following subsurface investigation guidelines are presented to acquaint the reader with methods and procedures utilized to determine the type and extent of foundation and borrow materials at a site.
Purpose

The purpose of the investigation is to define the foundation conditions under the proposed embankment and auxiliary works as well as gather information on materials of construction. The depth, thickness and areal extent of the geological units in the foundation and borrow areas will be identified from the investigation and the engineering properties of the soils in each unit estimated. The significant geologic units can be correctly identified by adhering to the following guidelines and the information contained in Section 2.3 "Geology". Information from subsurface investigations may assist in clarifying potential geologic hazards and confirming details for the geological damsite description.

Location, Number, and Depth of Test Holes

Up to 12 test holes may be required to investigate the foundation and borrow areas for a proposed small dam as shown on Figure 2.5.2. One hole should be drilled near the centre of the valley, one at or near the toe of each abutment and one between the proposed full supply level and top of the dam on each abutment. An attempt should be made to get all holes down to a depth at least equal to the maximum height of the proposed embankment. If granular, caving materials or very wet clays are encountered, this will not be possible with a hand auger and may represent "poor" conditions.

At conduit locations, holes should be drilled at the dam centreline and at the inlet and outlet locations. It should be noted that the centreline hole can double as a foundation hole. Hole depths should be 1 to 3 m greater than the maximum cut and preferably penetrate to competent materials.

In the earth cut or natural spillway locations, two or three holes should be drilled to a depth of 1 to 3 m below bedgrade elevation along the centreline to check if highly erodible materials are present and assist in defining soil stratigraphy.

If a concrete spillway is proposed, holes should be drilled at both ends of the spillway location and in the spillway inlet and outlet channels. The depth of holes should be 1 to 3 m greater than the cut required and preferably penetrate to competent materials.

In the selected borrow areas, holes should be drilled to a depth below the proposed depth of excavation. Generally, one hole in each corner of the selected area, with a check hole in the centre, is adequate. The number and depth of the holes required will depend on the suitability of the material for embankment construction purposes. The natural water content of the borrow materials should be near the plastic limit of the soil if possible. If the materials appear suitable and uniform, fewer holes are necessary than if borderline and variable materials are encountered. It should be noted that valley bottom borrow areas will likely be wetter than abutment borrow areas;
however, valley bottom deposits may be more variable and possibly less desirable.

For simple low height projects a minimum of four holes may be sufficient - one in the middle of the valley, one in each abutment and one in the auxiliary earth spillway or borrow area. The type and number of holes is up to the judgment of the project designer and is based on the size and complexity of the project, available soils information of the site, and information from the initial test holes.

**Sampling**

Samples should be taken from all test holes, classified and placed in watertight containers as soon as possible as it is particularly important that they remain at their natural water content. Ordinary quart sealers of the type used for home preserving make satisfactory containers of a suitable sample size. Care should be taken to ensure that the rubber ring is seated securely under the sealer lid. Plastic bags may be used providing they are tightly sealed and protected from damage.

Samples should be taken at 0.5 to 1 m intervals or at each change of material type to the bottom of the test hole. A hole log should be compiled for each test hole. The hole log should include the soil classification and depth of each material type as well as any other observations such as seepage, groundwater level upon completion, rocks or squeezing of the test hole. Figure 2.5.3 shows an example hole log.

Sampling of gravel pits is often required for determination of grain size distribution. The exposed vertical stratigraphic profile should be visually inspected and classified with the measured profile described in a similar manner to a borehole log. Samples should be taken at selected intervals which represent significant or typical layers in the profile. Care should be exercised to ensure the selected sample is not contaminated with material from other layers. Sand samples should be at least 4 L and gravel samples should be at least 20 L in volume.

Labels should be placed on each sealer or bag sample. The label should list: a) the project name; b) site number and land location description; c) the hole number (or the hole location); d) the depth the sample was taken; e) the date the sample was taken; f) a brief description of the sample; and g) if the sample is not at its natural water content, a notation should be made on the sealer label to this effect, i.e., "Sample dried out" or "Water added to drill hole", etc.

Soil testing can be carried out at the Geotechnical Division laboratory. If soil samples are sent for testing, additional information should include the overall purpose of the investigation as well as the intended purpose of each hole. ASTM Volume 04.08 describes sampling procedures in detail.
2.5.3 Types of Foundations

This section describes the general types of foundations and some of their related problems. Table 2.5.3 summarizes the classification of foundation and embankment materials for homogeneous small storage dams.

Impervious Foundations

Impervious foundations are characterized by clay soils with discontinuous layers of silt, sand or gravel. Wet clays are soft and may cause stability problems. During construction, the load of the embankment on the foundation may compress the soil structure and result in the development of excess pore-water pressure. If the pressurized porewater cannot drain freely, increased pore-water pressure can decrease the strength of the foundation as part of the embankment load is transferred from the soil particles to the water which has no strength. The most critical time for instability due to high excess foundation pore-water pressure during construction. High excess pore-water pressures can also occur within the embankment. As drainage slowly occurs, excess pore-water pressure dissipates and the embankment load is transferred entirely to the soil structure as equilibrium conditions are established. Consolidation settlement occurs as the load is transferred and the soil structure compresses. The water content of the soil decreases as drainage occurs.

Dry clays may have been precompressed by loads such as glaciers or overlying sediments which have been subsequently removed or desiccated by weathering processes. Settlement of a dry clay foundation will normally be much less than a wet clay foundation. Stability of a dry clay foundation will not likely be a problem. If dry clay foundations are cracked, there may be problems with seepage. Embankment cracking could occur due to excessive foundation settlement.
Pervious Foundations

Pervious foundations are characterized by their content of clean sand or gravel and to a lesser degree, silt. Problems are related to seepage losses and piping. Reservoir seepage through the foundation surface downstream of an embankment can dislodge fine sand and silt particles which are carried away in the seeping water. This type of erosion can progress towards the source of the seep and form a pipe beneath the embankment through which reservoir loss and embankment failure can occur. Piping failures occur as a result of the very small particles of silt and fine sand undergoing movement due to seepage in an unconfined state.

Stratified Foundations

Stratified foundations are characterized by alternate layers of impervious and pervious soils. Problems are related to seepage losses through pervious layers, piping through pervious layers if they are exposed downstream of the embankment and uplift pressures under the embankment. High excess pore-water pressures may develop in impervious layers but fairly rapid dissipation of excess pore-water pressures can also occur due to drainage into pervious layers.

Problem Foundations

Problem foundations include clay shale and sandstone, clays at very high or very low water contents, pervious soils, dispersive clays, organic clays and silts. Clay shales often contain bentonite layers which have very low strength. Even without bentonite layers, clay shales may have very low strength with subsequent stability problems. Precompression or weathering in clay shale and sandstone may have caused cracking through which seepage can occur. Sandstone deposits containing fine sand may be susceptible to piping. Clays at very high water contents may develop high excess pore-water pressures. Very dry loose soils may settle upon saturation due to reservoir seepage. Clays at very high or very low water contents will have water content control and compaction problems if used as embankment materials. Pervious soils and silts require special measures to control potential seepage and piping problems. Dispersive clays are chemically altered soils existing in a natural state which can erode in a similar manner to silt and fine sand. Organic clays and silts can develop seepage, settlement and stability problems.

2.5.4 Geotechnical Assessment

A geotechnical assessment is prepared by the project designer to determine the geotechnical suitability for construction of a dam at a potential dam site. The assessment is made based on the information collected from various sources during the geological and geotechnical investigations, obtained from field investigations, and contained in this manual. Table 2.5.3 provides a guideline for rating dams according to foundation and construction materials. The guideline defines the geotechnical suitability for construction of a dam based on
the classification of "Good, Fair, Poor". If the project cannot be assessed with confidence, it should be rated as "Poor" or a qualified professional engineer should be engaged to provide the necessary assistance or advice. Projects with a "Poor" geotechnical assessment cannot be designed using this manual but require the services of a qualified professional engineer for final design.

The "Geotechnical Assessment Report" is completed and signed by the project designer and forms an important input for project design.
AUXILIARY LABORATORY IDENTIFICATION PROCEDURE

NORMAL PLAGIOCLASE

(F)
Fibrous texture, color, shape, cleavage high index, common crystals, amounts of recognizable weathering, leaves, etc.

ADDITIONAL INCLUSIONS

49% or less passing 75μm (No. 200) Sieve
Run sieve analysis

PREDOMINANT PLAGIOCLASE

GEOGRAPHIC

Ferrugineous, ilmenite, olivine, silica, copper, etc. high index, common crystals, amounts of recognizable weathering, leaves, etc.

SCHEERITE

EXTRUSION

49% or less passing 75μm (No. 200) Sieve
Run sieve analysis

FLY ORENS

More than 75% passing 75μm (No. 200) Sieve
Run N₂ and N₂ on material passing 25μm (No. 40) Sieve

TABLE 2.5-2

<table>
<thead>
<tr>
<th>Normal Plagioclase</th>
<th>Normal Plagioclase</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Plagioclase</td>
<td>Normal Plagioclase</td>
</tr>
</tbody>
</table>

Note: Sieve sizes are U.S. Standards with sieve gradations.
Classification of Foundations and Embankment Materials for Homogeneous Small Storage Dams

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Water Content Variation from Optimum</th>
<th>Use as Foundation</th>
<th>Use as Impervious Borrow Material</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Suitability</td>
<td>Remarks</td>
</tr>
<tr>
<td>GC</td>
<td>1% dry to 3% wet</td>
<td>Good</td>
<td></td>
</tr>
<tr>
<td>SC</td>
<td></td>
<td>Fair</td>
<td></td>
</tr>
<tr>
<td>CL</td>
<td>1 - 2% dry or 3 - 4% wet</td>
<td>Poor Seepage</td>
<td>Poor</td>
</tr>
<tr>
<td>CI</td>
<td>Foundation is classified &quot;fair&quot; only if deposit is a valley unit less than 1m thick and underlain by a soil from one of the above groups. A 1-2 m key trench would be specified for foundation treatment.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CH</td>
<td>Poor Seepage and piping problem</td>
<td>Poor Seepage and piping problem</td>
<td>Water content control and compaction problems.</td>
</tr>
<tr>
<td></td>
<td>Outside range of 2% dry to 4% wet.</td>
<td>Poor May have stability problem due to high excess pore-water pressure if wet. May settle on saturation if dry and loose.</td>
<td>Poor May have stability problem due to high excess pore-water pressure if wet. Will settle on saturation if dry. Water content control and compaction problem.</td>
</tr>
<tr>
<td>OL</td>
<td>Poor May have stability, seepage, and/or settlement problems.</td>
<td>Poor</td>
<td>May have stability, seepage and settlement problems. Water content control and compaction problems.</td>
</tr>
<tr>
<td>OH</td>
<td>Poor May have stability and/or seepage problems.</td>
<td>Poor</td>
<td>May have stability and/or seepage problems. Water content control and compaction problems.</td>
</tr>
<tr>
<td>MH</td>
<td>Clay Shale and Sandstone</td>
<td>Poor Easily eroded.</td>
<td>Poor Easily eroded.</td>
</tr>
<tr>
<td>Dispersive</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

TABLE: 2.5.3
GRAIN SIZE DISTRIBUTION

PROJECT Example Only
LOCATION SAMPLED
DATE SAMPLED
SAMPLE NO
SAMPLED BY

87-12-22

D10 = 0.342 D30 = 1.461 D60 = 6.185
Cu = 18.110 D15 = 0.499 D85 = 15.280
Cc = 1.010

<table>
<thead>
<tr>
<th>COBBLES</th>
<th>GRAVEL</th>
<th>SAND</th>
<th>FINES</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>coarse</td>
<td>fine</td>
<td>coarse</td>
</tr>
<tr>
<td>0</td>
<td>8</td>
<td>38</td>
<td>20</td>
</tr>
</tbody>
</table>

Figure 2.5.1 GRAIN SIZE DISTRIBUTION CURVE

CLASSIFICATION
SW coarse to fine, medium predominant
46% gravel, 4% fines
Max. Size 33 mm
Figure 2.5.2 - Proposed Hole Locations for an Investigation of a Small Dam
PFRA TESTHOLE LOG

Project: John Q Public Dam Site

Site: Land location or description

Hole No.: T2
Elev.: 27.4 (assumed) m

Hole Location: Coulee bottom

Classified by: Klassen
Date: Sept. 18/87

Static Water Level: 0.9 m Date: Sept. 18/87

Test Installation: None

Page: 1 of 1 Comments: 100 mm Hand Auger

<table>
<thead>
<tr>
<th>Depths - m</th>
<th>Soil Classification</th>
<th>Soil Description, Characteristics, Abnormal Conditions, Sample Type, Water Data</th>
<th>Sample Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 0.2</td>
<td>Topsoil</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.2 - 0.5</td>
<td>C1 - Wp</td>
<td>alluvial, brown</td>
<td>Sealer #1</td>
</tr>
<tr>
<td>0.5 - 1.2</td>
<td>C1 + WP</td>
<td>alluvial, brown, squeezing</td>
<td>Sealer #2</td>
</tr>
<tr>
<td>1.2 - 1.3</td>
<td>SP</td>
<td>coarse, wet, seepage</td>
<td>Sealer #3</td>
</tr>
<tr>
<td>1.3 - 1.5</td>
<td>C1 + WP</td>
<td>till, brown, hard</td>
<td>Sealer #4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>stopped by rock @ 2.0 m</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Hole back filled</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>and staked</td>
<td></td>
</tr>
</tbody>
</table>

NOTE: All samples should be properly labelled with project, site, hole no., sample depth, sample classification and date.

Figure 2.5.3
GEOTECHNICAL ASSESSMENT REPORT

Project Title / Owner:  
Location:  
Designer:  
Date:  

Schematic foundation profile and site plan with soil logs and borrow area location on reverse side.

Foundation Conditions

<table>
<thead>
<tr>
<th>Location</th>
<th>Soil Types</th>
<th>Depth of Groundwater (m)</th>
<th>Potential Problems</th>
<th>Suitability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Valley Bottom</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Left Abutment</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Right Abutment</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Upstream</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Downstream</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Final Assessment and Comments:

Borrow Materials

<table>
<thead>
<tr>
<th>Location</th>
<th>Soil Types</th>
<th>Water Content</th>
<th>Optimum Water Content</th>
<th>Suitability</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Final Assessment and Comments:
2.6 SURVEYS AND MAPPING

Engineering surveys provide the information for determining the storage and areal extent of the reservoir, establishing the elevation and lengths of project components relative to the existing natural ground surfaces at the particular site location, and for checking previous work associated with hazard potential investigation.

Engineering surveys include surveys for topographical mapping of the reservoir area and project component locations up to the elevation of top of dam or greater, a profile of the stream bottom with valley cross-sections downstream from the embankment to the first private or public road crossing, a profile of the embankment location, and a profile and cross-sections of the earthcut spillway from the reservoir bottom to the confluence with the natural stream. Surveys shall be referenced to the provincial legal survey system and the Geodetic Survey of Canada where practical. Alternatively the surveys may be referenced to an existing prominent permanent landmark and/or benchmark with an assumed datum.

Surveys shall be performed by competent experienced personnel from the various area offices. This manual does not describe the equipment, methods or techniques for conducting engineering surveys. Reference is made to a survey manual prepared by PFRA.

After completion of the engineering surveys, topographic mapping is prepared of the project area. Mapping is drawn at a scale of 1:5 000 with a 1 m contour interval, however, other scales (1:4 000, 1:2 500) may be used depending on project size. Drafting equipment, methods or techniques are not discussed in the manual, however, reference is made to a manual prepared by the Conservation Service, "Drafting Standards Manual, December 7, 1989".

Area-Capacity Curves which specify the reservoir storage and flooded area with respect to elevation are prepared from computations based on topographical mapping or stream cross-sections. Area-Capacity Curves are plotted to include the top of dam elevation. An example of a manual computation for determining the information for plotting an Area – Capacity Curve is illustrated in standard drawing number SD01 "Area – Storage Capacity, Example Calculation Using Cross-Section Data" in Appendix A. It is noted that computer techniques may be available and utilized for producing Area Capacity Curves.

All work associated with engineering surveys, mapping and Area-Capacity Curves are to be documented in the appropriate field books, plans or computation sheets, signed, dated, and filed in the particular area office.
2.7 Site Investigation Guidelines

In order to ensure completeness, documentation, and an efficient practical sequence for project investigation the following guidelines are suggested. Also refer to Figure 1.4.2 Project Flow Chart.

1. Before any major investigations are initiated, an initial project assessment or reconnaissance level study is conducted. In discussion with the project proponent, design personnel determine the objective and requirements of the project, select potential sites utilizing available maps (1:50 000 NTS), aerial photographs, and a site inspection, estimate the potential water availability at the proposed sites, and estimates a preliminary cost based on similar types of projects. The potential water availability is estimated utilizing methods and procedures described in Section 2.4. The provincial water licensing authority is contacted to determine the availability of a water license for the proposed dam site. After completion of the reconnaissance studies, which are documented in a filed report, if the proposed project appears to be viable and the project proponent is interested, further investigation are justified and initiated on one or two of the most attractive site alternatives.

2. Hazard Potential Rating studies are performed, according to methods described in Section 2.2. If the Hazard Potential Rating is below specified limits, a technician may proceed with the project investigation and design. However, if the Hazard Potential Rating is above specified limits, a qualified professional engineer shall perform the project investigations and design. A Hazard Potential Rating Assessment report is prepared for site locations identified or recommended in the preliminary project assessment or reconnaissance study.

3. Geological and geotechnical studies are performed according to the methods described in Sections 2.3 and 2.5, respectively. Subsurface information is limited to available soil logs (water wells in general area), existing earth cuts (highways) and hand-augered holes or test pits excavated with local backhoes. If the foundation and earth materials are classified as "good" or "fair", a technician may proceed with the project investigations and design using the "Small Dam Design and Construction Manual". If the foundation and earth materials are "poor", the project investigations and design shall be completed by a qualified professional engineer. A geological damsite description report and a geotechnical assessment report are prepared for the site locations identified in previous studies.

4. Engineering field surveys are performed on the proposed sites identified in previous studies as detailed in Section 2.6. These surveys are normally conducted by the local area personnel. Topographical maps are prepared and Area-Capacity Curves are produced for the proposed sites from the survey information.
5. Hydrological studies including determination of the potential flood flows and a confirmation or review of the water supply availability for the proposed sites are conducted based on the procedures described in Section 2.4. A hydrology summary report is prepared for the site locations identified in previous studies.

After completion of the investigations, preliminary designs and cost estimates can be prepared to determine the feasibility of the proposed project. Occasionally further investigation may be required to clarify details or reconcile concerns discovered during project design.
3.0 DESIGN

3.1 GENERAL

The "Design Section" describes and summarizes the criteria, methods, procedures and standards for preparing preliminary and final designs for submission to provincial licensing agencies or for construction. One of the main attributes of this section is that the design details are arranged in a standard form such that design computations are minimal. For various parameters determined by project requirements, project investigations and site conditions, standard design details and/or drawings for project components are available. These standard details and drawings account for potential loading conditions and are pertinent over a range of component sizes.

It is necessary for the designer to select appropriate components and a project layout which provides the location and elevation of each component with respect to a site reference and each other.

Design inputs in terms of project requirements, site characteristics, and identifiable constraints dictate potentially feasible project concepts or strategies. Application of proven criteria, standards and methods provide a means of developing project alternatives for consideration. By developing project costs for the various project alternatives, a method is provided for selecting the most desirable or best project alternative. The designer must be aware of costs, safety, reliability and practicibility and exercise judgement and common sense in creating a project site layout and selecting project components.

In the past, many dam projects were designed and constructed based on a minimum of information on foundation and borrow material quality, structure performance and hazard potential evaluation. Also, many projects have been constructed with steep embankment slopes, inadequate or non existent filters, untested structures, and limited inspection or construction quality control. It has been noted that the overall performance of most of these projects has been satisfactory, however, a large number of these projects have a relatively lower safety factor compared to projects for larger dams constructed to accepted engineering practice.

This manual has attempted to update previous practices with present day state-of-the-art engineering practices and recent studies on dam safety. In doing so, it also attempts to reconcile the satisfactory performance of previously constructed projects which have been built with relatively little engineering input. The result is that the proposed designs and procedures incorporated into this manual have factors of safety somewhat less than normal safety factors established in engineering practice for large-scale projects. This implies that projects designed using this manual reflect a relatively higher risk with respect to performance and operation than for large-scale projects designed by professional engineering resources within PFRA. It is noted that these higher risk projects are also constructed at relatively low cost. The design procedures have been modified to provide a fairly high degree of security where it is hazardous, difficult or expensive to make repairs (such as in a stability
failure) while a lower degree of security is provided where repairs are not hazardous to the structure and are easily made, (such as for slope protection). This philosophy is in keeping with the desire to provide cost-effective structures for small projects.

It should be pointed out that constraints established for application of the procedures in this manual should not limit the development of potentially desirable sites. A site should not be eliminated because a dam would be higher than 8 m or have a storage greater than 400 $\text{dam}^3$. These sites should be considered with the assistance of a qualified professional engineer.

The operation of a water supply dam is different than a backflood dam. A water supply dam usually provides storage continuously throughout the year and subsequently develops a phreatic water surface within the embankment which saturates a portion of the embankment and its foundation. The temporary storage associated with backflood projects does not cause saturated embankment conditions. Also for a backflood project, the reservoir is empty before spring runoff and as such, the reservoir storage significantly reduces the reservoir outflow. The difference in operation between storage dams and backflood dams necessitates the establishment of specific design criteria for each in order to insure safe structures that can be constructed economically. In this section of the manual various design parameters and details which pertain either to a storage dam or backflood dam are identified and described.
3.2 DESIGN INPUTS

Before the sizes and details of project components can be determined, various types of information are required. This information is obtained from records of previous work performed in the area, from the results of preliminary planning or reconnaissance level studies, and from engineering investigations.

Reconnaissance studies provide the objectives and operational requirements of the proposed project, and result in the development of a project layout of applicable components for one or more site locations. Project objectives and requirements include: type of use (domestic, irrigation, etc.), location of point of use, required storage volume and delivery rate. A project layout shows the approximate position of an embankment, reservoir outlet, operating spillway and an earth cut overflow spillway.

The engineering investigation provides basic design parameters, site information and a basis for deciding as to whether the information contained in this manual can be used for final design. The following information should be available from the project investigations and become part of the design documentation:

2. Geological Damsite Description.
4. Topographic Surveys and Area Capacity Curves.
3.3 EMBANKMENT DESIGN

3.3.1 Description

The dam embankment is the project component which provides the barrier or confinement for capturing and storing runoff from the watershed basin above the location of the dam. The embankment also provides a freeboard or safety allowance. Freeboard between the top of the embankment and the full supply level of the reservoir contains the effect of wind generated waves and provides an allowance for an increase in reservoir elevation during passage of floods through the spillway system.

Embankments considered in this manual are normally homogeneous earth fills constructed from locally available soil materials. The upstream slope is normally protected from wave erosion with rock and/or granular slope protection. The downstream slope is protected from wind and runoff erosion by a grass cover. Embankments may incorporate embankment seepage control measures consisting of an internal granular filter system.

Depending on annual inflows and demand, the water level in the reservoir may vary from empty to FSL. During periods of above average runoff, the reservoir level may remain close to FSL throughout one or more years which results in saturation of the embankment and surrounding foundation. The embankment designs in this manual make provision for these saturated conditions and steady state seepage conditions which result from full saturation.

3.3.2 Criteria and Standards

Engineered embankments are designed to provide adequate factors of safety against failure for end of construction conditions, long- term steady state seepage and rapid drawdown of the reservoir. The selection of the appropriate factor of safety is based on the adequacy of the site investigation and soil testing program, the complexity of the site, and consequences of failure. Small storage dams with fair or good foundations and fair to good borrow materials can be designed on the basis of general guidelines, standards, and criteria which can be expected to provide an adequate factor of safety against failure with respect to the risk normally associated with this type and scale of project.

In general, geotechnical design criteria are directed towards three basic areas of concern; stability, seepage and erosion. Design options are presented for some embankment components such as upstream slope protection. Judgement is required to select the appropriate option considering all factors involved including inspection and maintenance. An external component such as upstream slope protection can be monitored for performance and repaired if damaged. The foundation must be competent to support the embankment load and the embankment slopes must be internally stable. The embankment and foundation must have an adequate factor of safety for the worst loading condition to which they might be subjected. Seepage through the embankment and foundation must be controlled to prevent significant loss of stored water, internal erosion of
embankment or foundation materials (piping), and instability of the embankment or foundation. Erosion of the upstream and downstream slopes from wave action and surficial runoff respectively, must be prevented.

The following design details are based on PFRA experience and the experience of others with the performance of small storage dams with fair to good foundation and borrow materials as outlined in Table 2.5.3.

**Crest Width**

The crest must be a minimum of 3 m. Greater widths to accommodate construction and farm equipment or provide roadway access may be required. Commonly 5 m is used to allow for possible erosion along the crest shoulders.

**Downstream Slope**

A minimum inclination of 2.5 or 2 horizontal to 1 vertical is required for stability purposes. Minor slope sloughing requiring maintenance has developed on some 2:1 slopes so 2.5:1 slopes are recommended for long-term stability. Slopes of 2:1 may be employed on low embankments less than 2 m in height. Vegetative cover or a gravel blanket to prevent erosion by surface runoff is recommended. Native grasses can be planted on the downstream slope which has been topsoiled using material previously stripped from the embankment foundation.

**Upstream Slope**

A minimum inclination of 3 horizontal to 1 vertical is required for stability purposes. Flatter slopes may be employed to reduce slope protection requirements depending on the availability and cost of construction materials.

**Key Trench**

A minimum trench base width of 3 m is required to accommodate earth excavation and compaction equipment. A minimum depth of 1 m will usually remove the majority of weathered foundation materials; however, the trench depth should be extended to the base of any pervious, fractured or crumbly materials encountered during key trench excavation. A project in which unsuitable foundation materials are still encountered at a trench depth of 2 m must be halted and assessed by a qualified professional engineer. Side slopes of 1:1 are usually adequate to ensure good bonding with the key trench backfill. The key trench should be located along the embankment centreline or upstream of centreline. In addition to a key trench, the foundation area of the dam embankment is stripped of all vegetative material and topsoil. Topsoil may be stockpiled and later placed on embankment slopes and grassed for erosion protection. After foundation stripping, the foundation is examined in detail to insure that it can be classified as "good" or "fair" in accordance with the criteria specified for use in this manual. A key trench is not usually required for backflood projects depending on the type of foundation materials.
Foundation Seepage Control

Dams which require foundation seepage control measures are associated with poor foundation conditions. As this manual is only applicable to good or fair conditions, no information on foundation seepage control is provided. However, it is noted that the key trench described above does provide protection from foundation seepage due to shallow surficial fractured, crumbly or pervious foundation materials. Dam projects which require foundation seepage control measures (with poor foundations) are beyond the scope of this manual.

Embankment Seepage Control

Internal granular filters control and collect seepage through the embankment and consist of horizontal and vertical or inclined segments. The horizontal segment without the vertical or inclined segment requires a well compacted, uniformly dense fill to effectively lower the long-term steady state piezometric surface through the embankment. The horizontal filter will not control seepage that may occur along layers or cracks in the fill. The inclined or vertical segment is not essential for well-constructed embankments with good quality control on relatively incompressible foundations; however, an inclined or vertical filter segment provides a high degree of safety against slope instability and internal erosion or piping when adequate control over material quality or construction methods can not be assured over the entire construction period. If the inclined or vertical segment is omitted due to lack of suitable granular materials or high costs, additional effort should be made during construction to ensure the embankment is well compacted with good water content control and is free of any layers.

A decision to omit the inclined or vertical segment should be based on understanding and accepting an unknown degree of risk associated with the omission versus the additional cost of supplying and placing filter material.

Three possible configurations for an internal filter system are shown on Figure 3.3.1. Configuration A shows a horizontal filter which is easy to construct and should be included in all earth dams greater in height than 2-3 m. Configurations B and C show inclined and vertical segments in addition to the horizontal filter. These two configurations have the advantage of intercepting seepage occurring along layers or cracks in the fill. Layering of the fill can easily occur during construction if unsuitable equipment is used, borrow materials vary in character or consistency, construction operations are not closely monitored or if the effects of unfavourable weather conditions are not rectified. Cracking of the embankment can occur from excessive post construction settlement due to inadequate compaction of the fill or deformation of a soft foundation as well as desiccation during periods of low reservoir level. These configurations of the vertical or inclined segments markedly improve the stability of the downstream slope and prevent problems with embankment seepage. Horizontal finger drains reduce the quantity of filter material required.
The selection of the most appropriate configuration of the vertical segment is dependent on site specific conditions. Inclined segments may be particularly appropriate for sites utilizing Zone 2 embankment materials. As inclined filters present some construction difficulties, vertical segments installed by trenching through previously placed fill may be more appropriate for homogeneous embankments. Additional construction details are given in Section 4.2.

Inclined or vertical filter segments are recommended for all earth dams, especially those with soil conditions not categorized as good; however, they are not considered to be essential for low hazard dams constructed according to recommended procedures. Owners may feel the additional costs are more important than the benefits; however, the owner should be aware that there is a degree of risk which depends on factors not easily controlled or definable prior to development of a seepage or stability problem. Additional costs may be incurred after completion of construction to remedy problems. There is also a possibility that complete failure could occur. In this respect, the inclusion of internal filters is a form of insurance which should be evaluated in terms of both probability and consequences of failure versus the cost of remedial measures or complete replacement.

There are many dams existing today which are performing satisfactorily without internal filters of any kind. Those with reservoirs which are full only part of the time may take an extremely long time to develop steady state seepage problems, but problems may also develop during the initial reservoir filling or a few years after construction. Minor seepage losses may be acceptable if erosion does not threaten the safety of the structure. The possibility of remedial measures such as surficial granular blankets over seepage areas to prevent piping or berms to improve stability may be acceptable compared to the certainty of the expense of internal filters during construction.

The final choice is up to the owner, but it falls upon the designer to explain all associated risks, costs and benefits so that an informed decision can be made. Horizontal filters are required for all embankments greater in height than 2-3 m. Vertical or inclined segments are also recommended, especially for dams with fair foundation and borrow materials where limited or indeterminate inspection and quality control may occur during construction. Internal granular filters are not usually required on backflood projects depending on the type of foundation materials.

**Upstream Slope Protection**

Standard designs for upstream slope protection have been developed for this manual based on the design procedures and criteria used for large engineered projects while taking into account the relatively small reservoirs, a somewhat higher risk of failure which may be acceptable, and the case of which maintenance associated with slope protection for small dam projects may be performed by the Owner.

Slope protection provides resistance to erosion from wind-generated waves. The erosion potential of the waves is dependent on the intensity and duration of the wind and the unobstructed length of
reservoir over which the wind travels.

Most small dam reservoirs are long and narrow with an average length-width ratio in the order of 10, and are usually less than 1 km in length although some reservoir lengths may be up to 1.6 km. The approximate effective fetch for this shape of reservoir corresponding to reservoir lengths of 1 km and 1.6 km is 300 m and 500 m, respectively.

Some embankments are sheltered from the effects of the prevailing winds. For these embankments, the major axis or lengths of the reservoir is not in line or parallel to the direction of the prevailing winds. For convenience and for use of Table 3.3.1, this condition is defined as "Exposure Condition A". Embankments which face the prevailing winds and considered to be unsheltered are defined as "Exposure Condition B". In this case the length or main axis of the reservoir is in line or parallel to the prevailing winds. Geographically the winds in southwest Alberta are stronger than in the other areas of the prairies. Therefore, embankments which face the prevailing winds and are located in southwest Alberta are defined as "Exposure Condition C".

Table 3.3.1 describes design standards for the above exposure conditions with respect to three methods of slope protection: riprap; gravel; and unprotected clay till slopes. The design standards in Table 3.3.1 are generally applicable to all small dam reservoirs up to 1.6 km in length. The reservoir length is a straight line distance away from the embankment. Upstream slope protection for reservoirs which are longer than 1.6 km or relatively wide (length - width ratio less than 5) should be designed to a more refined criteria used for large engineered projects or based on the reference "Upstream Slope Protection for Earth Dams in the Prairie Provinces" by N. Peters and J. E. Towle (1979).

The determination of minimum design upstream slope for the slope protection method "unprotected embankment slopes", utilizing the side slope ranges specified in Table 3.3.1, is based on prorating the appropriate range with respect to reservoir length to the nearest 0.5 slope rate. A project with a reservoir length of 1.45 km under exposure condition B would have a minimum unprotected embankment slope of 8:1 based on the following computation:

\[
6 + \left[ \frac{(1.45 - 1) \times (9 - 6)}{1.6 - 1} \right] \approx 8.25 \text{ or slope of 8.0:1}
\]

The granular materials specified in Table 3.3.1 should be reasonably well graded with no sizes lacking and no excess of material in any size range. In this regard rock riprap should have smaller rocks filled in around larger rocks with no voids where bedding gravel is exposed to wave action.

The upstream slope protection required for a backflood dam would normally consist of a grassed 3:1 slope. Periodic maintenance may be required under unusual severe conditions. In order to provide for some protection and reduce potential maintenance, the upstream slope may be designed as for storage dams described above.
Other materials, such as concrete block revetment system, plastic grid networks, and concrete-filled jute sacks, have been used successfully on various engineering projects by owners and agencies other than PFRA. Alternate upstream slope protection may be considered for small dam projects provided approval and design details have been obtained from a professional engineer.

3.3.3 Methods

The methods described in this section provide direction to the project designer for selecting an appropriate cost effective embankment section including embankment geometry and configuration, foundation preparation, internal drainage systems and slope protection.

1. Confirm that the criteria, methods, and standard in this "Small Dam Design and Construction Manual" are applicable for the embankment design of the proposed project. Application of the "Small Dam Design and Construction Manual" requires that:

: the maximum height and the reservoir storage at the top of dam elevation is equal to or less than 8 m and 400 dam$^3$, respectively

: the project hazard potential is low

: the foundation and borrow materials are good or fair; and

: the project design is being performed by experienced, trained, competent personnel.

2. Confirm availability and quality of embankment materials including fill, filter materials, bedding gravel, riprap and pit run gravel for alternative sites identified in reconnaissance investigations.

3. Select embankment details based on the cost and availability of materials; and owners operation requirements and assessment of risk. Details include: crest width; side slopes; key trench; internal filter systems and slope protection. The most economical upstream slope protection alternative (riprap, pit run gravel, or flat embankment slopes) is selected for project design. When considering a number of site alternatives it may be convenient to compare upstream slope protection cost on an area (square metre) or lineal (per metre) basis.
4. Determine optimum dam orientation at each proposed site by considering various embankment alignments in conjunction with locating the auxiliary earth spillway. Using the top of dam elevation established from freeboard considerations in Section 3.4.2 and topographic mapping, plot the location of the dam and calculate earthwork quantities. Prepare an estimate of cost for all earthwork items using a local assessment of unit prices. The optimum orientation minimizes embankment quantities and provides an earthwork balance between the embankment fill and auxiliary earth spillway excavation. Select the most economical site.

5. All design parameters, dam location and embankment details are reviewed. Design sketches of the embankment cross-section, dam profile are prepared. An embankment cross-section at the maximum section should show the following geotechnical details:

- Embankment zones if applicable,
- Inclination of the upstream and downstream slopes,
- Crest width and elevation,
- Type, thickness and extent of upstream slope protection with elevation of FSL,
- Configuration of internal filter system, and
- Foundation design details include cutoff or key trench depth, width and sideslopes. (It should be noted on the construction drawings that the limits of trench excavations are subject to change in the field depending on the nature of the foundation material.)

Figure 3.3.2 shows an embankment design cross-section example. All work is documented using computation sheets.
Table 3.3.1  Design Standards for Upstream Slope Protection

1. Riprap

<table>
<thead>
<tr>
<th>Embankment Slope 3:1</th>
<th>Reservoir Length (km)</th>
<th>Exposure Condition</th>
<th>Riprap Thickness (mm)</th>
<th>Average Rock Size (D50) (mm)</th>
<th>Bedding Gravel Thickness (mm)</th>
<th>Bedding Gravel Effective Size (D85) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>less than 1</td>
<td>A</td>
<td>200</td>
<td>130</td>
<td>150</td>
<td>38</td>
<td>38</td>
</tr>
<tr>
<td>less than 1</td>
<td>B</td>
<td>250</td>
<td>170</td>
<td>170</td>
<td>38</td>
<td>38</td>
</tr>
<tr>
<td>less than 1</td>
<td>C</td>
<td>300</td>
<td>200</td>
<td>200</td>
<td>38</td>
<td>38</td>
</tr>
<tr>
<td>1 - 1.6</td>
<td>A</td>
<td>200</td>
<td>130</td>
<td>150</td>
<td>38</td>
<td>38</td>
</tr>
<tr>
<td>1 - 1.6</td>
<td>B</td>
<td>300</td>
<td>200</td>
<td>200</td>
<td>38</td>
<td>38</td>
</tr>
<tr>
<td>1 - 1.6</td>
<td>C</td>
<td>400</td>
<td>260</td>
<td>260</td>
<td>38</td>
<td>50</td>
</tr>
<tr>
<td>greater than 1.6</td>
<td>(professional engineering assistance)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

2. Gravel

Average Stone size D50 for a 5:1 embankment slope is 40 mm

Average Stone size D50 for a 6:1 embankment slope is 25 mm

<table>
<thead>
<tr>
<th>Reservoir Length (km)</th>
<th>Exposure Condition</th>
<th>Gravel Thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>less than 1</td>
<td>A, B, C</td>
<td>400</td>
</tr>
<tr>
<td>1 - 1.6</td>
<td>A</td>
<td>400</td>
</tr>
<tr>
<td>1 - 1.6</td>
<td>B</td>
<td>500</td>
</tr>
<tr>
<td>1 - 1.6</td>
<td>C</td>
<td>600</td>
</tr>
<tr>
<td>greater than 1.6</td>
<td>(professional engineering assistance)</td>
<td></td>
</tr>
</tbody>
</table>

3. Unprotected Embankment Slopes

<table>
<thead>
<tr>
<th>Reservoir Length (km)</th>
<th>Exposure Condition</th>
<th>Slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>less than 1</td>
<td>A</td>
<td>3:1 to 5:1</td>
</tr>
<tr>
<td>less than 1</td>
<td>B</td>
<td>5:1 to 7:1</td>
</tr>
<tr>
<td>Less than 1</td>
<td>C</td>
<td>7:1 to 9:1</td>
</tr>
<tr>
<td>1 - 1.6</td>
<td>A</td>
<td>4:1 to 6:1</td>
</tr>
<tr>
<td>1 - 1.6</td>
<td>B</td>
<td>6:1 to 9:1</td>
</tr>
<tr>
<td>1 - 1.6</td>
<td>C</td>
<td>9:1 to 14:1</td>
</tr>
<tr>
<td>greater than 1.6</td>
<td>(professional engineering assistance)</td>
<td></td>
</tr>
</tbody>
</table>

Note: Unprotected embankment slopes are only applicable to embankments constructed with materials classified as "good" (till). The erosion resistance (self armouring) of compacted till is variable depending of the stone content. Unprotected slopes require inspections and special inspections after storms when reservoir levels are high. Maintenance consisting of the placement of gravel armour in eroded areas may be required.

Erosion of the steeper slopes within the allowable range will be more likely than on flatter slopes.
NOTE — Finger drains incorporated as shown minimize material quantities. The finger drain should extend across the lowest portion of the coulee bottom with a minimum width of about 10 metres.

CONFIGURATIONS FOR INTERNAL FILTER SYSTEMS

Figure 3.3.1
- LEGEND -

1 IMPERVIOUS - 1% dry to 3% wet of optimum water content
2 RANDOM - Spillway excavation
3a GRANULAR - Select filter sand
3b GRANULAR - Select bedding gravel
3c GRANULAR - Slope protection alternative 40 mm average size
4 ROCK RIPRAP - 200 mm average size

PROPOSED EMBANKMENT DESIGN CROSS-SECTION EXAMPLE

Figure 3.3.2
3.4 SPILLWAY SYSTEM DESIGN

3.4.1 Description

A spillway system provides the means by which excess water is conducted past the dam embankment from watershed runoff after the reservoir behind the dam has reached the full supply level (FSL). If a spillway system was not included in the project, water levels may rise and overtop the dam, causing varying degrees of damage to the embankment including complete wash out or failure of the dam. Therefore, the spillway system is a protection device. The level of protection is dependent on the capacity of the spillway system. The larger the spillway, the higher the flood flows (the lower the frequency of occurrence) which can be passed before overtopping and the higher the level of protection. The spillway system capacity and associated flood surcharge (the water depth above FSL for operation of the spillway) is considered in determining the top of embankment elevation.

There are numerous styles and types of spillway concepts and devices. Each spillway type has advantages and disadvantage for a given set of project conditions and constraints. For the convenience of the users of the manual, several standard spillway structures are included in Appendix A. Also Table 3.4.1 shows a range of discharges for various types of standard structures which provide satisfactory performance.

A spillway system generally comprises one or more spillway components. Each spillway component is designed to provide capacity (ability to pass inflow floods) for different operation or flood conditions. Spillway components are normally defined as the operating (principal) spillway component or the auxiliary (emergency) spillway component. However, in certain situations both the operating and auxiliary spillway functions are combined into one spillway structure.

The operating spillway is normally constructed of durable, dependable, low maintenance materials such as concrete, wood or steel and may constitute a significant portion of the overall project cost. The function of the operating spillway is to provide capacity for passing frequent flood flows which occur during most years and result from low to average annual precipitation (average spring snowmelt runoff and average rainfall) and watershed base flows from groundwater springs. Operating spillways are designed to pass the design flow with satisfactory hydraulic performance and no damage to the structure, outlet channel or embankment.

The auxiliary spillway is generally an earth-cut channel situated in one of the abutments. It would normally provide a source of earth material for construction of the embankment. Therefore, depending on the volume of materials associated with the embankment and earth-cut channel the incremental cost of the auxiliary spillway may be relatively small. The function of the auxiliary spillway is to provide a channel around the dam for passing flood flows in excess of the operating spillway capacity. It would be sized to discharge up to and including the selected project design flood without overtopping the embankment.
An auxiliary spillway operates intermittently for short durations throughout the project life. Depending on the flows and erosion resistance of the soil, damage to the earth-cut channel could be expected to occur during operation and the earth-cut channel may require periodic maintenance involving the replacement of eroded material.

The concept of a spillway system utilizing a low-maintenance, small-capacity structure to carry normal runoff flows and an inexpensive earth-cut spillway to carry the infrequent larger runoff flows, but which would require occasional extensive maintenance, provides a cost-effective, practical approach to flood protection over the life of the project. Other spillway concepts involving higher or lower risks with associated savings or increase in costs may be appropriate in particular situations depending on site conditions, project requirements and owner preferences.

### 3.4.2 Criteria

The design of a spillway system is based on a set of standards or criteria developed from hydraulic and structural theory and practice, and tempered with the experience from the performance of previous spillway designs. An appropriate set of criteria ensure uniform, safe, cost-effective, successfully performing spillway structures.

Directly related to spillway design is the provision of freeboard on the embankment which allows safe reservoir rise above FSL to provide the head necessary to pass flood flows through spillway systems. Freeboard also protects the embankment from overtopping due to wind and wave action.

### Location

An operating spillway structure is located within the embankment section, either adjacent to the abutment or along the valley floor depending on the required operating elevations and foundation conditions. All concrete structures are placed on undisturbed material (in an excavation). Structures of other materials may be placed on fill material which has received a specified foundation treatment. Local areas of soft alluvial or pervious foundation material should be avoided for structure locations.

The auxiliary earth-cut spillway which is excavated into a relatively erosion resistant material suitable for embankment construction, is located in a natural low area, gully or natural run, or through an upland area but at least 3 m from the embankment (from cut stake of earth cut channel to fill stake of embankment). During operation of an earth cut spillway, erosion takes place starting at the downstream end of the excavated channel and progresses towards the reservoir. The rate of erosion is dependent on the discharge rate, duration of use, velocity of flow in the channel and erosive susceptibility of the channel bed. The location of the earth-cut spillway should be selected to provide a sufficient length to allow for erosion without jeopardizing the storage of the reservoir.
Type

The type of structure generally selected is based on hydraulic capabilities, economics, structural limitations, and project owner preferences. Table 3.4.1, describes and indicates the relative range of hydraulic capacity for various types of structures that have provided satisfactory performance. The hydraulic criteria which results in satisfactory performance of the Standard Structures in "Appendix A" are based on engineering hydraulic model tests and existing successful designs.

Size

The physical size (length, width, height) of a spillway is based on providing a discharge capacity for a given level of flood protection. Based on the PFRA Dam Safety Program which is applicable to all PFRA dams with a maximum height equal to or greater than 8 m or a storage at the top of dam elevation equal to or greater than 60 dam$^3$, the minimum design flood is the 1:100 flood event. This design flood is referred to as the Safety of Dam Design Flood (SDDF) and the project spillway system must have sufficient capacity to pass the peak instantaneous discharge with a nominal freeboard allowance. For dams with a storage at top of dam elevation less than 60 dam$^3$, the SDDF recommended in this manual is the 1:50 flood event. Implicit in the selection of the SDDF is the requirement that the project hazard potential is "low" ("C"). No reservoir-routing effects are taken into account when selecting the spillway design flows. In situations where there is a large amount of reservoir surface area, the designer may have a reservoir flood-routing analysis performed by a qualified professional engineer and subsequently base the spillway design flow on the reservoir outflows. Inherent with the refinement of using a flood-routing approach for determining reservoir outflow capacity requirements is the need to perform similar hydrological refinements beyond the scope of the manual in assessing inflows into the reservoir in order to keep the level of analysis consistent throughout the project evaluation. The spillway system capacity should include the contribution from all spillway components.

The frequency of the design flood for an operating spillway cannot be conveniently defined for the variations of dam projects to which this manual is applicable. Selection of the operating-spillway design flood or even the need for an operating spillway for a particular project must be based on the knowledge and judgement of the designer, the policies and objectives of funding agencies and the project owners preferences.

In general terms, the operating spillway reduces maintenance costs and extends the life of the auxiliary earth spillway over the life of the project but may signifi cantly increase project cost. There may be situations where a relatively large-capacity, expensive operating spill- way is considered appropriate and other situations where an operating spillway may be considered unnecessary. In these projects where an operating spillway is deemed unnecessary, the project spillway requirements would be satisfied by the earth-cut spillway. In all cases the earth cut spillway should be seeded to an erosion-resistant grass.
In order to provide some assistance to the designer in selecting the appropriate operating spillway for a given project a list of guidelines is described below:

: Dam sites which have base flow should be provided with an operating spillway.

: Relatively large expensive projects which may require potentially high maintenance costs should be provided with an operating spillway.

: Small projects with relatively low potential maintenance costs may be considered not to require an operating spillway.

: Dam sites, which provide relatively flat, naturally grassed overflow areas (or constructed grassed earth-cut spillways) and produce non-erodible velocities for the Safety of Dam Design Flood, may be considered not to require an operating spillway.

The minimum design flood for projects requiring an operating spillway is the 1:2 frequency flood. The design flow is the instantaneous peak inflow without considering reservoir flood routing effects. The operating spillway must provide capacity to pass flows from the design flood and the natural stream base flow where it occurs. The natural base flow (springs) is quantified and added to the flood flows calculated in the Hydrology Section to provide a design discharge for sizing the operating spillway.

Spillway structures like the CSP Drop Inlet Spillway require certain operating conditions at the outlet section of the structure to ensure satisfactory performance of the outlet energy dissipater. For the CSP Drop Inlet Spillway, the outlet invert elevation of the conduit must be above the plunge pool water elevation, that is, not affected by the downstream tailwater conditions. When the water surface elevation of the existing stream is known for the given spillway design discharge (usually from a downstream tailwater analysis), the structure outlet and exit channel elevation and dimensions can be selected to provide conditions for satisfactory structure performance. In lieu of a tailwater analysis the following procedure may be used to select the exit channel bed elevation and dimension, and spillway conduit outlet invert.

1. Based on field observations, select a reference elevation in the existing stream adjacent to the dam site which would be representative of the water surface elevation for approximately a 1:2 flood event. This reference elevation would typically be an average spring flood water mark; or the bank full capacity of the stream. The designer must judge the most appropriate reference elevation for each specific site.

2. Set the exit channel bed elevation one conduit diameter below the reference stream elevation.

3. Provide an exit channel with 0% bed slope, 3:1 side slopes and a bed width equal to the plunge pool bed width.
4. The exit channel bed elevation provides the basis for setting the structure outlet elevation.

For structures with design discharges greater than the 1:2 flood event, a higher reference stream water elevation or a tailwater analysis may be required.

The selection and design of an operating spillway for a backflood dam is also primarily based on the understanding and judgement of the designer. Backflood projects cover a wide range of dam heights, reservoir sizes and levels of investment. Backflood projects which involve ditches and dyking to constrain or trap moisture from snow accumulation in sheltered or depressional areas, cultural practices like terracing, and drainage of sloughs do not normally justify an operating spillway. Other backflood projects located on natural, well developed drainage channels require operating spillways or control works to satisfy riparian requirements and to drain flooded areas to prevent crop damage due to summer precipitation. In the case where an operating spillway or control works is required, it is sized to meet the following conditions:

- Pass the 1:2 flood events with reservoir at FSL neglecting reservoir-routing effects.

- Drain the reservoir in two (2) days assuming structure is operating at maximum discharge continuously.

It is noted that a spillway sized for the above condition would have the capability for passing an actual or effective flood event (inflow flood frequency) considerably greater than the 1:2 event when taking into account reservoir storage.

The design flow for an auxiliary spillway is the difference between the project "Safety of Dam Design Flood" flow and the flow of the operating spillway at the maximum flood surcharge elevation.

The inlet invert of the earthcut auxiliary spillway is set at the surcharge elevation of the operating spillway. With inlet set at this elevation, the auxiliary spillway only discharges when the operating spillway exceeds its design capacity. For projects with no operating spillway, the inlet invert of an earth-cut spillway would be set at FSL. The drawings SD 03A and SD 03B "Auxiliary Earth Channel Spillway" in Appendix A enables the designer to consider various auxiliary spillway widths and flow depths for optimizing the spillway system and the embankment fill requirements. The details of drawings SD 03A and SD 03B are based on having the auxiliary spillway separated from the embankment, providing low flow velocities during operation; and locating the spillway outlet such that its operation will not damage the embankment.

The design floods specified in these criteria and summarized in Table 3.4.2 are minimum requirements which provide flood protection for a level of risk considered appropriate by current engineering practice for the size and hazard potential associated with project design based on this manual. Higher floods may be considered when a project proponent desires an increased level of flood protection to offset
future maintenance costs or in considering an optimal cost relationship between spillway size and embankment height.

**Strength**

The standard structures presented in "Appendix A" have been designed by professional engineering resources within PFRA based on current engineering practices with the underlying philosophy of simplicity in materials, details and construction methods. No structural criteria or methods for structural design are provided in this manual as all spillway structures are to be selected from the standard designs contained in "Appendix A". In certain situations which require unique or non-standard structures, a qualified professional engineer should provide advice and/or the design and drawings.

**Freeboard**

Freeboard is an additional height of embankment above the design reservoir water elevation. Freeboard accounts for the consolidation of fill materials and increased water levels due to the effects of the weather. The top of dam elevation is determined by adding the freeboard allowance to the design reservoir elevation. Two conditions are to be considered when calculating freeboard in determining the top of dam elevation. The case which yields the highest top of dam elevation is used for the design.

For a reservoir elevation at the full supply level (FSL) a freeboard allowance is based on wave action, settlement of embankment materials, and vertical cracking due to weathering (frost action or desiccation). The freeboard allowance in this case is termed "normal freeboard allowance" and can be determined from the graph in Figure 3.4.1. The portion of the "normal freeboard allowance" due to wave action was determined by methods developed by the U.S. Army Corps of Engineers and Savile and is based on a wind velocity of 100 km/hr, a ratio of the maximum straight unobstructed reservoir length to the average reservoir width equal to 1.0, a wave height equalled or exceeded by 2% of the waves, and an upstream embankment slope of 3:1 complete with riprap. A maximum straight line unobstructed reservoir length and average reservoir width ratio of 1.0 defines an approximately square-shaped reservoir. For this case, the effective fetch which relates the reservoir shape to wind-generated wave action is approximately 0.9 times the unobstructed reservoir length. The settlement was based on 2% of the embankment height measured to the top of dam. A constant height of 0.4 m accounts for the effects of frost action and desiccation. The graph in Figure 3.4.1 shows a plot of Freeboard Allowance (m) vs. Maximum Height of Dam (m) for a family of curves with a straight line reservoir length of 0.5, 1.0 and 2.0 km. Interpolation between the curves is required for other reservoir lengths.

For a reservoir elevation at the maximum design flood surcharge level or the reservoir elevation during passage of the SDDF, a freeboard allowance is based on embankment settlement and a nominal allowance of 0.15 m. This freeboard allowance is termed the "flood freeboard allowance" and can be determined from the graph in Figure 3.4.1.
3.4.3 **Standard Spillway Structures**

A collection or catalogue of standard spillway structures are enclosed in "Appendix A". These designs have been prepared by the Development Service based on current engineering practices but recognizing the requirement for simple construction materials and methods, and the relatively short life/high risk nature of privately owned and operated projects. The appendix includes a description, list of advantages and disadvantages, and a complete construction drawing showing design details. Standard specifications for construction of the standard structures are included in "Appendix B".

The construction drawings are complete and may be used for project approval or construction. Design details which are a function of project size are shown in tabular or graphical form. For situations where a standard spillway is inappropriate, a special spillway design is to be performed by a qualified professional engineer.

3.4.4 **Methods**

The methods described in this section provide direction to a project designer for selecting the type, size, and cost of spillway structures required for a complete spillway system and provide the associated construction drawings. Refer to flow chart in Chart in Figure 3.4.2.

1. Confirm that the criteria, methods and standard structures in the "Small Dam Design and Construction Manual" are applicable for the spillway design of the proposed project. Application of the manual for spillway design requires that:

   : the maximum height and the reservoir storage at the top of dam elevation is equal to or less than 8 m and 400 dam$^3$, respectively;

   : the project hazard potential is low (C);

   : the foundation and borrow materials are good or fair; and

   : the project design is being performed by experienced, trained, competent personnel.

2. Select the design flood and determine the flood flows utilizing the methods and procedures described in Section 2.4 Hydrology. The criteria used in this manual provides a minimum Safety of Dam Design Flood (SDDF) of 1:100 or 1:50 depending on reservoir storage and a minimum Operating Spillway Design Flood (OSDF) of 1:2. The design flood discharge for an auxiliary spillway is the difference between the SDDF and the capacity of the operating spillway.

3. Select a spillway system concept (operating spillway and auxiliary earth-cut spillway); locate the orientation and elevations of the spillway structures and determine applicable standard drawings of spillway components for the design flood requirements (OSDF, SDDF); determine freeboard allowances and the top of dam
elevations; and estimate approximate project costs.

4. Consider alternative spillway concepts (operating spillway components or auxiliary spillway widths) and repeat Step 3. After a number of alternatives have been studied, the least cost alternative is selected for final design if all other factors are equal.

5. All design parameters, and the selection of location, type, and size of structure are reviewed. The appropriate standard spillway drawings are selected from Appendix A. A site layout drawing is prepared. The dimensions and other construction parameters are added in the appropriate places in copies of the standard drawings. All work is documented using computation sheets, summaries and drawings.
Table 3.4.1 General Guide to Spillway Selection

<table>
<thead>
<tr>
<th>Spillway Type</th>
<th>Maximum Discharge (m³/s)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>CSP Drop Inlet Spillway</td>
<td>4.3</td>
<td>Low maintenance, small capacity spillway for dam height from 3 - 8 m.</td>
</tr>
<tr>
<td>Reinforced Concrete Stoplog Outlet</td>
<td>14</td>
<td>Small capacity operating spillway or outlet structure which does not accommodate ground elevation changes. Used normally in backflood schemes. Structure height available on standard drawing 1.5 or 2.0 m.</td>
</tr>
<tr>
<td>Earth Channel Spillway</td>
<td>Flow limited by width and flood surcharge depth.</td>
<td>Large capacity spillway which accommodates unlimited drop height. Used primarily as an auxiliary spillway in conjunction with an operating spillway. This spillway requires maintenance depending on frequency of operation and its resistance to erosion.</td>
</tr>
<tr>
<td>CSP Sloping Pipe Spillway</td>
<td>3.8</td>
<td>Low maintenance, small-capacity spillway for dam height from 3 - 8 m. This spillway requires a relatively large headwater to pass the design discharge.</td>
</tr>
<tr>
<td>Rock-Armoured Overflow Spillway</td>
<td>14</td>
<td>Spillway with moderate flow capacity and maintenance requirement for full range of dam heights.</td>
</tr>
</tbody>
</table>
Table 3.4.2  Design Floods

<table>
<thead>
<tr>
<th>Maximum Dam Height (m)</th>
<th>Maximum Storage (S) (dam³)</th>
<th>Safety of Dam Design Flood (frequency)</th>
<th>Operating Spillway Design Flood (frequency)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>S&lt;60</td>
<td>1:50</td>
<td>Minimum 1:2</td>
</tr>
<tr>
<td>8</td>
<td>60&lt;S&lt;400</td>
<td>1:100</td>
<td>Minimum 1:2</td>
</tr>
</tbody>
</table>
NOTE

*L* is unobstructed Reservoir Length

FIGURE 3.4.1 FREEBOARD ALLOWANCE
Figure 3.4.2. Flow Chart for Spillway Design
3.5 RESERVOIR OUTLET DESIGN

3.5.1 Description

A reservoir outlet structure provides for a controlled release or withdrawal from the reservoir over a selected range of reservoir depths. Normally an outlet structure releases water to a downstream area of the same natural channel occupied by the embankment and reservoir. A typical reservoir outlet structure would consist of an upstream pipe, a gatewell including a gate and hoist, a downstream pipe, and riprap plunge-pool energy-dissipater. The structure would be located through the embankment.

Another type of outlet structure commonly utilized consists of a pressure water supply pipe through the embankment with a buried water works valve complete with value stem extension and a riprap plunge-pool energy-dissipater. This type of outlet may be connected to a pumped or gravity water supply line.

Projects with relatively low embankment heights such as a backflood irrigation schemes may utilize a stoplog style of outlet structure.

Construction drawings for standard outlet structures are compiled in "Appendix A - Standard Drawings".

3.5.2 Criteria

The design of the reservoir outlet structures was developed from criteria and standards based on standard engineering practice, water works standards and provincial government guidelines. Criteria govern the location, size and strength of the structure.

Location

An efficient outlet structure provides for the release of water over the greatest range of reservoir water depths practical. Therefore, the structure invert elevations are placed at approximately the same level as the stream bottom. This minimizes the reservoir dead storage (volume of water supply not available or accessible for use) and maximizing the reservoir live storage (volume of water supply which is available or accessible for use). It is noted that for projects utilizing a pump delivery system, the live storage is referenced to the pump intake structure. The inlet to the outlet works must be set higher than the existing stream bottom to prevent plugging from debris and silting.

The structural components are placed on undisturbed soil (in an excavation) with good or fair foundation characteristics. This is normally achieved by locating the structure in the floodplain off to the side of the stream channel or at the edge of one of the embankment abutments. An inlet and outlet ditch is excavated to join the structure with the stream bottom.
A gatewell which houses a slide gate and hoist is located upstream and adjacent to the top of embankment which minimizes the length of pipe under reservoir water pressure, localizes the pressure section, and provides convenience for operation and maintenance.

A water works valve on a pressure pipeline is located in the downstream portion of the embankment section with a minimum earth cover of 2.5 m. The earth cover provides protection from freezing and enables maintenance with small equipment.

The reservoir outlet structure is not located adjacent to a spillway structure so as to minimize the effect of a failure of one structure on the other.

**Size**

The size of an outlet structure depends on the required rate of withdrawal and available head at the structure. The withdrawal rate is based on the use of the water, the operation of the structure as perceived by the owner-operator, and any riparian considerations (based on provincial licensing requirements). The head available at the structure varies from FSL to the crown of the inlet pipe section.

**Strength**

The standard structures compiled in "Appendix A" have been designed by professional engineering resources within PFRA based on current engineering practice and utilize standard materials and construction procedures. Therefore no structural criteria are provided in this manual. Construction drawings for standard outlet structures are described in "Appendix A - Standard Structures".

3.5.3 **Standard Reservoir Outlet Structures**

A collection of standard outlet structures is enclosed in "Appendix A". Like the standard spillway structures, the designs have been prepared by professional engineering resources within PFRA based on current engineering practice but recognizing the shorter project life and higher risk associated with this type of owner-operator project and utilizing standard methods and construction materials. The appendix includes a descriptive list of advantages and disadvantages and complete construction drawings showing design details. Specifications for these structures are included in "Appendix B - Specifications in Construction".

The construction drawings may be used for project approval and construction. Design details which are a function of project size are shown in tabular or graphic form. A qualified professional engineer should be consulted to provide special designs and construction drawings for special situations where the standard drawings are not applicable.
3.5.4 Methods

The methods described in this section provide direction to a project designer for selecting type, size, and cost of a reservoir outlet structure complete with construction drawings.

1. Investigate, determine, and discuss with project proponent the necessity for an outlet structure. An outlet is always necessary when downstream riparian releases must be provided. The provincial water-licensing authority provides information on riparian requirements. An outlet structure is desirable and cost effective when a continuous downstream water demand is to be satisfied. However, when downstream demands are intermittent and of short duration, it may be economical to provide water from the reservoir by pumping or siphons. Also, no outlet structure is required if the water is to be used only for sprinkler irrigation adjacent the reservoir. The owners may have a preference on a future requirement which should be considered in the design of the outlet structure.

2. Locate the structure in order to provide a maximum range of reservoir operating levels and good foundation conditions.

3. Determine the required release rate based on demand conditions or riparian considerations.

4. Select the most cost-effective standard plan which best satisfies the demand requirements. Discharge rating curves are shown on the standard drawings.

5. All design parameters, structure location and structure size are re-checked by the designer. Copies of pertinent standard outlet drawings are obtained and pertinent design parameter included in the site layout drawing. All work is documented using computation sheets, summaries, and drawings.
3.6 DESIGN OUTPUT

Upon completion of the design (preliminary and final) and selection of appropriate pertinent structure component plans, an overall location plan and site layout drawing is prepared. Also a set of specifications and an operation and maintenance manual are compiled from the standard specifications included in Appendix B and the information in Section 5 - "Operation and Maintenance". The "Project Location, Layout, and Details" drawing should include the following information:

1. Project name.

2. Location map showing project location relative to major town or city.

3. Legal description and ownership of land which the project will affect.


5. Site layout (topographic map) showing extent of reservoir relative to legal boundaries, location of dam and component structure.

6. Embankment cross-section, dam profile, auxiliary earth-cut spillway profile and cross-section, and soil logs.

7. Reservoir area-capacity curve.

8. A spillway-system rating curve.

The results of the investigations and design can now be packaged or compiled together with standard drawings, and specifications for:

: Review and approval by a qualified professional engineer or other designated PFRA technical authority;

: Engineering report to owner describing project details, costs and benefits;

: Application for approval for construction and water licence; and

: Documents for construction by owner or contractor.

1. Project Review and Approval

For purposes of review and approval within PFRA, the project designer prepares a review package including:

(a) Copy of request from project proponent and summary of project demands.

(b) Copy of design inputs - Hazard Potential Assessment, Geological Damsite Description, Geotechnical Assessment, Hydrology Summary Report.
(c) Copy of drawings (layout and structures) and specifications.
(d) Copy of cost estimate.
(e) Copy of design summaries or other information as required by the reviewer.

2. Engineering Report to Owner - Client

For purposes of providing the project proponent with a description of the proposed project and provide him with information to base a decision for proceeding with the project, the project designer prepares a brief engineering report including:

(a) A written description of the project requirements, design features, estimated costs and benefits, and applicable grants or loans available under PFRA programs.

(b) Copy of drawings (layout and structures) as required.

3. Application for Approval for Construction

The project designer files an application with the appropriate provincial agency for construction on behalf of the project proponent and includes copies of design drawings (layout and structures).

4. Construction

After the project proponent has confirmed his intention to proceed with the project, provincial approval for construction and a water license has been obtained, and funding has been arranged, the project designer prepares a set of documents including drawings and specifications for construction by the owner or contractor. During construction, the designer confirms design assumptions and monitors work. At the end of construction, as-constructed drawings are prepared and an operation and maintenance manual is prepared and provided to the project owner.

5. Application for Water Licence

After project construction the project designer files a water licence application with the appropriate provincial agency for the project on behalf of the project proponent.
4.0 CONSTRUCTION

4.1 GENERAL

The information in this section pertains to construction methods, and quality control methods and testing applicable to the supervision of construction of water-retaining facilities.

Depending on the program, construction of small storage dams may be undertaken by local earth-moving contractors normally involved with projects other than water-retaining structures. These contractors may not recognize the importance of the requirements described in the drawings and specifications, so where a program assigns responsibility for construction supervision to the designer the methods and procedures applicable to a particular project should be discussed in detail with the contractor prior to the contractor accepting the work. Quality control by visual evaluation assisted with in situ soil testing as required ensures that the final condition of the project is consistent with requirements and assumptions used in the design.

Depending on the program, the project proponent may be responsible for undertaking the project construction. It is important that the proponent understands and is able to follow the required construction methods or procedures and has sufficient and appropriate equipment and resources to carry out the construction. Shortcomings in these areas may result in construction deficiencies, wasted time and wasted funds, particularly if certain construction operations have to be redone. In cooperation with the project proponent the project designer is encouraged to pursue a practical, efficient, and pertinent construction inspection program.
4.2 EARTHWORK

Construction materials for the various embankment zones, foundation preparation, placing procedures, and compaction requirements are described in the standard specifications in Appendix B. Additional information on internal drainage systems and compaction is provided in this section. Clay materials, Zone 1 and Zone 2 depending on the amount of mixing, compaction and moisture content, are used for homogeneous embankments. Suitable materials of minimum quality from required excavations can be effectively utilized in upstream and downstream areas. In general, the higher-quality, suitable materials (close to optimum water content) should be placed towards the central core, grading to the minimum-quality, suitable materials at the outer limits of the embankment cross-section.

Internal Drainage Systems

Various configurations of internal drainage systems are shown on Figure 3.3.1 and described in Section 3.3.2. The selection of the most appropriate configuration is based on a number of factors including availability of materials and the preferred method of construction. The inclined or vertical segment of the internal filter may be constructed in a number of different ways. A trenching approach for vertical segments can be used after impervious materials have been placed and compacted. It is important to re-establish contact with the buried surface of the filter zone. This method minimizes interzone intrusion and quantity overruns; however, there may be problems with segregation when dumping widely graded sand and gravel filter material into a trench. Segregation results in layers or streaks of very coarse material along edges or boundaries. For this method of construction, relatively uniform filter material is recommended to minimize segregation. Relatively uniform material would be within the medium sand to fine gravel size range. The depth of trenching should not exceed about 2 m and the trench should be backfilled immediately after excavation.

A step method is widely used for construction of inclined segments where the construction surface elevation of the upstream zone, the filter zone and the downstream zone are staggered as shown on Figure 4.2.1 (A). When the elevation of the upstream zone surface is higher than the filter zone surface by a maximum of 1 to 2 m, the sloping surface of the upstream zone is bladed off to expose a compacted dense surface in a neat line. This shoulder trimming can be accomplished with a backhoe or by backblading with a dozer. The horizontal surface of the filter zone can then be cleaned off with a dozer blade or grader. Filter material can then be placed against the trimmed upstream zone slope with a front end loader and compacted as required. This method can result in some intrusion of the filter zone into the downstream zone; however, except for quantity overruns this is not considered serious.

Figure 4.2.1 (B) shows a construction method where the filter zone is higher than the adjacent embankment surfaces while Figure 4.2-1 (C) shows the filter zone lower than the adjacent embankment surfaces. Both of these methods can result in uncompacted zone boundaries which may
have a significant effect on the effectiveness of the seepage control when it is placed in service. In addition, significant interzone intrusion can result in large quantity overruns or effective filter widths less than required. These two methods are not considered as effective as the method shown in Figure 4.2.1(A); however, both methods provide satisfactory internal drainage systems.

**Compaction Equipment**

Sheepsfoot rollers are the best type of compaction equipment for earth dams due to the mixing and kneading action of the protruding feet which assists in obtaining a uniform soil moisture content and bonding successive lifts to form a uniformly dense embankment without layering along which seepage or sliding could occur. Sheepsfoot rollers can be used for essentially all clay soil types.

Alternate compaction equipment may be used providing each compacted lift is scarified to a depth of at least 50 mm to prevent layering and at least 95% of Standard Proctor maximum dry density is obtained. Alternate equipment to sheepsfoot rollers generally includes rubber-tire rollers and padfoot or wedgefoot rollers. These compactors can all produce satisfactory densities when the soil moisture conditions are near optimum; however, thorough mixing of the fill materials is required before compaction. The availability of alternate equipment may influence the final choice of compaction equipment. Compaction by routing earth-moving equipment may not produce a consistent and uniform degree of compaction. This construction practise should be avoided.

Rubber-tire rollers are available in many sizes and weights with a corresponding range of maximum tire pressures. The pressure exerted on the soil is proportional to the tire inflation pressure which, in conjunction with the number of passes and lift thickness, affects the embankment density obtained.

Rubber-tire rollers and padfoot or wedgefoot rollers apply a pressure at the soil surface which leaves the surface smooth and hard. This type of surface is good for drainage of rain to minimize water content increases due to infiltration; however, the surface should be thoroughly disced or scarified to a depth of at least 50 mm prior to additional fill placement. Layered surfaces within an embankment are undesirable because seepage may occur along the layers. Weak layers may also be created along which sliding could be initiated.

Padfoot or wedgefoot tamping rollers have short feet with large end areas in comparison to sheepsfoot rollers. Tamping rollers with large foot end areas do not blend and mix embankment materials as effectively as sheepsfoot rollers with tamping foot end areas of 4500 to 6500 mm². The tamping feet of sheepsfoot rollers penetrate the upper loose layer and the load of the roller is initially carried partly by the lower layer and partly by the upper layer on which the drum surface is riding. The penetration of the feet through the upper layer fuses the upper and lower layers into a homogeneous dense mass. As the upper layer becomes denser and stronger, the feet will not penetrate as far and the drum
lifts off the soil surface or "walks out". This causes maximum pressures to be transferred to the upper portion of the lift by the tamping feet; however, the churning action of the feet leaves the fill surface in a rather rough condition (roller walkout may or may not be visible). After compaction, the rough surface is suitable to receive the next lift without additional surface preparation. Experience has shown that adequate compaction can be obtained in a variety of soils whether the rollers walk out or not.

Since the unit pressure on the soil is low at first and only gradually increases as the soil becomes strong enough to support the tamping feet and the rollers begin to walk out, sheepsfoot rollers can compact efficiently over a wide range of water contents. Surface compactors such as rubber tire rollers cause the density to be sensitive to changes in water content. The sheepsfoot roller feet are effective for breaking down clods of soil; however, rocks in the fill can cause the roller to lift so that the soil adjacent to the rock may not be well compacted. Therefore, rocks larger than 150 mm should be removed from the fill before compaction.

Due to the churning action of the feet, slightly wet soils can be dried by rolling and slightly dry soils can be sprinkled while rolling. In gravelly clay soils such as clay till, the feet of sheepsfoot roller wear down due to abrasion and must be built up by welding. The end area of the feet require regular checks to ensure that they are not worn down below the specified requirements.

Unit foot pressure required for satisfactory densities generally range from 2400 kPa to 2800 kPa although pressures as low as 1725 kPa can produce satisfactory densities under appropriate conditions. The unit foot pressure is calculated by dividing the total weight of the roller by either the total area of the maximum number of tamping feet in one row parallel to the axis of the roller, or by 5% of the total foot area, whichever is greater. The applicable foot pressure is calculated in this manner to account for the possibility that there may be many rows of feet with only a few feet in each row and therefore foot pressure calculated on the basis of one row of feet will show very high foot pressures. In this situation there would in fact be many more feet in contact with the soil due to the large number of closely spaced rows of feet.

In general, the foot end area of sheepsfoot rollers should be near the lower end of the specified range of 4500 to 6500 mm² for clays of high plasticity and near the higher end of the specified range for silty cohesive soils to achieve the best density results. In addition, soils with water contents wet of optimum are best compacted at foot pressures near the lower end of the specified range, especially if these soils are of high plasticity.

Ten passes of a sheepsfoot roller are generally adequate to obtain satisfactory density and mixing. Complete coverage of the lift ensures uniform density throughout.
In summary, rubber tire rollers can compact more efficiently with less passes than sheepsfoot rollers; however, this benefit is offset by scarification requirements and difficulties with water content control to attain the required degree of compaction. Sheepsfoot rollers can compact a variety of soils over a range of moisture contents to produce a homogeneous dense embankment. In addition, the churning action of the feet assist in drying a slightly wet soil or mixing sprinkled water into a slightly dry soil.
INCLINED FILTER CONSTRUCTION METHODS

Figure 4.2.1
4.3 QUALITY CONTROL INSPECTION FOR EARTHWORK CONSTRUCTION

Visual evaluation and quality control in situ testing during construction are necessary to ensure that the embankment is constructed in accordance with the requirements called for during the design stage. Visual evaluation involves the inspection of soil types, moisture content and excavation sequence at the borrow source as well as observations of the operation of compaction and hauling equipment. Visual evaluation performed by experienced personnel and followed with remedial actions when required forms the essential part of construction control necessary for a relatively homogeneous and dense embankment. In situ testing basically involves the determination of the field density and water content.

The field density and moisture content of a compacted soil is an indirect measure of other significant soil properties. Maximum dry density is obtained when the water content is at its optimum value as determined by the Standard Proctor test. Field dry density is compared to Standard Proctor maximum dry density as a means of quality control. The optimum water content for a low to medium plastic soil is near its plastic limit.

Density and water content test results supplement the visual evaluation of the embankment construction and provide the inspector with guides to judgment. Experience at estimating the water content of a soil relative to its optimum water content in addition to knowledge of borrow material characteristics gained during preconstruction investigations improve the inspector's ability to visually evaluate embankment construction. Testing frequency during the initial construction stages to confirm the inspector's ability at visual evaluation can often be reduced as the work progresses.

The amount of testing required varies with the inspector's experience in quality control and types of specifications set up for earthwork.

PFRA construction specifications often utilize a method type of specifications to control the characteristics of the fill. With method specifications, the Contractor follows specified methods and procedures using particular equipment which are known to produce a homogeneous and dense embankment. The emphasis with method specifications is on the establishment of a proper sequence and pattern of material and equipment utilization at an early stage of construction. Limited density testing is carried out to confirm that adequate densities have been obtained.

End-product specifications require the Contractor to achieve a specified degree of density by whatever means he can. Equipment types and construction techniques are not specified. With end-product specifications, very close construction supervision is required throughout the construction period. Frequent in situ density tests are carried out to determine if density and moisture requirements are being obtained. Test results are used to correct construction operations.
The following summaries regarding the apparatus and procedures of common quality control tests are described in detail in Volume 04.08 of ASTM Standards.

**Volumeter or Rubber Balloon Test Method - ASTM D2167**

This test method may be used to determine the density of natural or compacted soil or soil-aggregate mixtures. It is not recommended for soft, deformable soils or soils with crushed rock fragments or sharp edge materials which may puncture the rubber membrane. The apparatus, shown on Figure 4.3.1, consists of a calibrated membrane vessel containing a liquid within a relatively thin, flexible elastic membrane designed for expanding into and measuring the volume of an excavated hole beneath a rigid base plate.

The procedure to determine the density is as follows:

1) Prepare the surface at the test location so that it is reasonably smooth and lay the base plate on this surface.

2) Place the rubber balloon apparatus on the base plate, open the air valve until the balloon has completely deflated against the surface of the soil and take the initial reading on the volume indicator.

3) Remove the rubber balloon apparatus from the base plate and excavate a hole through the base plate removing all loose particles and saving all soil in an airtight container for future mass and water content determination.

4) Place the rubber balloon apparatus on the base plate, open the air valve to allow the balloon and fluid to fill the hole and take the final reading on the volume indicator.

5) The difference between the initial and final readings is the volume of the hole.

6) Determine the wet and dry mass of the soil excavated from the hole and use these values to calculate the wet and dry density.

In using this method, care should be exercised to have the inside of the pit smooth with rounded corners so that the balloon will completely fill the excavated hole and not bridge over small pits and corners in the surface. This method is similar to the sand cone method which utilizes standard Ottawa sand for determination of the volume of the hole. Another variation is to use wheat rather than Ottawa sand. These two methods are not used as extensively as the rubber balloon method.

**Nuclear Gauge Test Method - ASTM D2922 & D3017**

Nuclear gauge test methods utilize radioactive sources to determine moisture content and density of natural or compacted soil or soil-aggregate mixtures. Nuclear methods are rapid, nondestructive methods which require trained operators who are familiar with routine safety
precautions and applicable government regulations regarding radioactive materials.

The nuclear densometer is a relatively expensive device which measures moisture content and density indirectly by measuring the return of gamma-rays or neutrons emitted from a radioactive source probe after they have passed through the soil. There are several ways of performing the test, the most common of which for construction involves insertion of the radioactive probe into the soil. Regular calibration of the machine against a test block of known density is required.

The advantage of the nuclear test method is that results are provided within minutes which eliminates delays for a contractor waiting for approval to place and compact another lift. However, correlation curves for wet density and water content supplied by the manufacturer may not be accurate for all soil types. Correlations to wet density as determined by the volumeter should be checked for the soil types at a specific project if an accurate determination of wet density is required. In addition, water contents do not correlate very well with water contents determined in an oven, particularly with highly plastic or organic soils. Alternate methods of rapid water content determination (microwave oven, speedy moisture meter) may provide accurate information and avoid the use of the nuclear densometer for water content determination. Other disadvantages are the high cost of the equipment, safety requirements for handling radioactive equipment and time delays for shop repairs if breakdown occurs.

Standard Proctor Test Method - ASTM D698

The purpose of this test is to determine the relationship between moisture content and dry density of a soil compacted in a standard manner and to determine the maximum dry density and optimum moisture content for the soil. The maximum dry density can be compared to field dry density test results to determine if the field densities are adequate. Similarly, field moisture contents can be compared to optimum moisture content.

The apparatus consists of a standard cylinder mold of the dimensions shown in Figure 4.3.2 having a volume of 944 cm³. The mold is provided with a machined base which forms a bottom for compacting soil in the mold. A removable extension fastens to the top of the mold to hold the soil during compaction.

The compacting is done with a standard rammer having a milled face 51 mm in diameter with a mass of 2.49 kg. An arrangement is provided for lifting the rammer exactly 305 mm and allowing it to free fall to the surface of the soil.

The compaction of the soil in the standard test is made in three equal lifts with 25 blows of the rammer for each lift. When the cylinder has been filled and struck off level with a straight edge at the top, the mass and water content of the compacted soil are determined. By means of these quantities, the dry density can be computed. In a similar manner the dry density is determined for successively damper mixtures of the soil, usually in increments of 2% moisture
content, until the dry density after compaction decreases conspicuously with increasing moisture content. A curve is plotted showing the relation between dry density and moisture content. The optimum moisture content according to the standard Proctor test is the value of moisture content at which the dry density is a maximum. Figure 4.3.3 shows a typical standard Proctor curve.

One-Point Proctor Field Test

During the construction of an embankment the inspection staff have often experienced difficulty in selecting the appropriate Proctor compaction curve to be used for comparison with the field density test because an insufficient number of compaction curves were available. The choice of appropriate compaction curve is generally based upon visual classification of the soil and/or Atterberg limit tests. Even well trained, experienced personnel can make errors in selecting the maximum density and optimum water content.

A one-point Proctor field test is used by PFRA to determine values of Standard Proctor maximum dry density and optimum moisture content quickly and easily in the field using a Standard Proctor dry density determined at the field moisture content on one sample only. The process requires base information shown on Figure 4.3.4 which was established on the basis of many laboratory standard Proctor test results on samples occurring on the Prairies. These curves are of a general nature and more accurate curves for a specific site may be developed by combining a number of standard Proctor curves from the site.

The one-point Proctor dry density and water content are plotted on Figure 4.3.4 and the variation from optimum water content determined from the curves. The optimum water content of the sample is the sum of water content of the one-point Proctor test plus the variation from the optimum water content. Enter the chart with the optimum water content and determine the standard Proctor maximum dry density at the intersection of the optimum water content and the line defining the dry density water content at zero variation from optimum.

If the one-point Proctor results fall on the wet side of optimum the chance of error may be quite large because of the close spacing of the lines wet of optimum water content. To overcome this problem, the soil should be dried to optimum or below optimum water content prior to conducting the one-point Proctor test. This will minimize the error in determining the maximum dry density and optimum water content.

Water Content Determination

Density testing in the field requires an accurate determination of water content, although it may not be necessary where variation from optimum water content can be estimated visually by experienced personnel. Where power is available at a construction site and suitable equipment and facilities are available, water contents can be determined by conventional oven or microwave oven. Conventional oven water content determination test methods are described in ASTM D2216 while microwave methods are described in ASTM D4643. Water contents determined by microwave oven have a fairly close correlation to those
determined with a conventional oven. A general procedure, established experimentally by PFRA for prairie soils and recommended for use, is similar to the procedure for a conventional oven with some notable exceptions. Soil specimens in pyrex glass dishes are placed in the oven on the glass bottom plate and heated for 15 minutes. After this time a dish of water is also placed in the oven and the oven turned on for a further 20 minutes. The water dish prevents the glass bottom plate from cracking. The dry mass of the samples can be determined after they have cooled.

A rapid test method for determination of moisture in soils is by the use of a calcium carbide gas pressure moisture tester (ASTM D4944); commonly called the speedy moisture tester. In this test calcium carbide and a soil sample along with two steel balls are placed in a container and shaken for 1 to 3 minutes. The calcium carbide reacts with the free moisture in the sample producing acetylene gas and heat. The steel balls assist in breaking down cohesive materials. After the test a dial reading on the container and a conversion chart are used to determine the moisture content. This test is physically demanding and is not recommended if many water content determinations are required.
VOLUMETER FIELD DENSITY TEST APPARATUS

Figure 4.3.1
Extension

Volume 944 cm³

Compaction Cylinder

For 305 mm fall

Mass 2.49 kg

Compaction Cylinder
(Standard Mold)

STANDARD PROCTOR LABORATORY TEST APPARATUS

Figure 4.3.2
TYPICAL 5 POINT STANDARD PROCTOR CURVE

Figure 4.3.3
**Example:** The one-point Proctor dry density and water content of 1.52 t/m³ and 18% respectively are plotted on the chart as Point A which falls on the line of 4% below optimum water content. The water content is increased by 4% to the optimum of 22% at Point B. The Standard Proctor maximum dry density of 1.58 t/m³ at Point C is obtained at the optimum water content curve immediately above Point B. The field dry density can then be compared to the Standard Proctor maximum dry density to obtain a percentage.

Figure 4.3.4
4.4 CONCRETE

Construction details and material requirements are shown or described on the standard drawings in Appendix A and the standard specifications in Appendix B.

Usually it is economical and convenient to obtain concrete from a local redi-mix plant. However, satisfactory concrete may be batched on-site using pre-packaged mixes or processed local aggregate materials.

Methods for production, placing, finishing, and curing and quality control of concrete are described in "Concrete Materials and Methods of Concrete Construction", CAN3-A23.1-M90 by the Canadian Standards Association and "Design and Control of Concrete Mixtures", by the Canadian Portland Cement Association.
4.5 CONSTRUCTION SUPERVISION AND QUALITY CONTROL TESTING

The following list provides a guideline for the minimum level of inspection, testing, and supervision for a typical project:

1. Upon completion of stripping and key trench excavation the foundation area is inspected to ensure that all organic material and top-soil has been removed and that weathered material in key trench has been removed. The classification of the foundation material as "good" or "fair" is also confirmed.

2. Upon completion of foundation preparation, the foundation is inspected. The location and elevation of structures are confirmed.

3. Before components of structures are assembled, they are checked for size.

4. Before structures are backfilled, they are checked for alignment, assembly and elevation.

5. During concreting, slump, air content and strength tests are performed as specified by CSA:CAN3-A23.2-M90, "Methods of Test for Concrete". The frequency and number of tests for projects with concrete volumes over 50 m³ are specified in Section 17 of CSA:CAN3-A23.1-M90, "Concrete Materials and Methods of Concrete Construction". For projects with concrete volumes less than 50 m³, as is the case for the majority of small dams, the following guidelines should be followed:

   - at least one compressive strength test per pour;
   - a minimum total of 3 compressive strength tests per project;
   - one slump test and one air-content test shall be made with every strength test; and
   - each strength test shall consist of a minimum of two standard cylinders, tested at 28 days. A third cylinder, tested at 7 days, is desirable for the purposes of early estimation of the 28 day strength.

6. During embankment construction, compaction quality-control is performed by visual evaluation, supplemented with in situ testing as described in Section 4.3 and Appendix B.

7. Upon completion of the construction and before issuance of a certificate of completion, an inspection is conducted to ensure that all details of the contract have been fulfilled.

8. Detailed records should be kept of all activities, observations, and decisions made during construction. Daily written records with sketches and photographs should be made of difficulties, problems, and sequence of events that lead to design changes during construction.
4.6 AS-CONSTRUCTED DRAWINGS

During construction notes or sketches are prepared by the project inspector on any revisions or changes to drawings. At the end of construction, the drawings are revised to reflect any changes during construction.

The as-constructed drawings then accompany an application for a Water Rights license sent to the province by the owner.
5.0 OPERATION AND MAINTENANCE

5.1 GENERAL

The determination, definition and description of the operation, maintenance and inspection requirements and procedures are generally the final formal responsibility of the designer. The effective application of operation, maintenance and inspection procedures ensures safe, cost-effective performance during the life of the project. The normal method of conveying and explaining the procedures for operation, maintenance and inspection for dam projects is by a project operation and maintenance manual. The manual is provided to the owner and can be considered as a component of the project. This section describes the elements which are considered and included in an operation and maintenance manual. The end of this section contains standard clauses and formats which can be used for preparing an operation and maintenance manual for individual projects. A set of "As-Constructed" drawings is included in the operation and maintenance manual.

At the beginning or in a "General" section of an operation and maintenance manual, the purpose of the manual should be explained, the project location specified, the project components identified and described, and the documentation of project performance explained. Also included in this section should be a summary of pertinent project data including component sizes and capacities, a map of the drainage basin and a key plan.

It is noted that the operation and maintenance manual can be considered as the final design task and results in the final design output.
5.2 \textbf{OPERATION}

For "normal operation" the water withdrawals and release schedules are specified. Riparian demands may be required as a condition for approval and licence but may not be specified quantitatively in terms of a release rate and schedule. During "normal operation" all withdrawal rates, duration, and reservoir water levels are recorded in the Project Log. The Project Log is a sheet that lists all operation, maintenance and inspection events as they occur throughout the life of a project. All events are dated and summarized briefly.

All spillway structures are operated automatically and therefore require no adjustments or control during passage of a flood event. The reservoir is not utilized for flood control and knowledge of flood inflows is not required. During "flood operation" the spillway operation should be documented by recording the duration and reservoir water levels.

It should be emphasized that under no circumstances should the reservoir FSL be increased with flashboards or small temporary dikes.
5.3 MAINTENANCE

This section should distinguish between two classes of maintenance: preventative maintenance; and special maintenance.

Preventative maintenance is performed on a regular or scheduled basis to various project items which experience normal wear and tear due to project operation and weather. A sheet included in the manual known as the "Preventative Maintenance Schedule" describes the project items, maintenance requirements and time for service.

Special maintenance may be required if a defect is discovered in the project or an unusual condition occurs which threatens the safety of the dam. In these situations, technical assistance should be obtained to determine the significance of the problem and possible solution or repairs.

All maintenance should be documented in the Project Log.
5.4 INSPECTION

This section should be separated into two portions: routine inspection; and semi-annual and special inspections. All inspections including comments are documented in the Project Log.

Routine inspections are performed when the operator is in the project area. Each component is quickly inspected by the operator using a check list for reference. The inspection and comments should be recorded in the Project Log.

One of the semi-annual inspections is performed when the reservoir levels are at the yearly maximum and the spillway system is operating which generally occurs during the spring runoff. The other semi-annual inspection is conducted in the autumn when the water levels are low. The inspections are performed by the owner using a checklist for reference. Special inspections are performed by the operator after any severe or unusual weather conditions such as a heavy summer rainstorm or tornado.

A list of adverse and potentially dangerous conditions are listed in a table. The designer should include this information in the operation and maintenance manual for reference.
5.5 OPERATION AND MAINTENANCE MANUAL MODEL

Enclosed at the end of this section is an outline for production of an operation and maintenance manual. An operation and maintenance manual can readily and easily be produced for a particular project by compiling, deleting or modifying the various clauses in the blanket manual which are pertinent to the particular project. A complete set of "As-Constructed" drawings are included with the operation and maintenance manual.

The contents of the blanket manual are based on guidelines used by PFRA for preparation of operation and maintenance manuals for large engineered dam projects.
1.0 GENERAL

1.1 Purpose

The purpose of this manual is to provide the project owner-operator with a set of instructions and information prepared by the designer for the operation, maintenance and inspection of the dam project. With the knowledge and application of the manual the project should provide safe, cost effective performance throughout the project life for the conditions and assumptions used in the design.

It is the responsibility of the project owner to implement the procedures specified in the manual. The conscientious application of this manual represents an implied condition of project approval and for granting of a water licence and unsafe operation of the project may cause the water licence to be revoked and the dam breached.

1.2 Location

The John Doe Dam Project is located on Sometimes Dry Creek in the SW1/4-6-26-16-3 approximately 10 km west and 6 km south of the Town of Vanguard, Saskatchewan.

The project location and access is shown on the Key Map.

1.3 Description

The project was constructed in 1988 by John Doe to provide water storage for irrigation of 160 acres adjacent to the reservoir. The project is situated on a small creek with "good" foundation conditions and has a "low" hazard potential.

The embankment is a homogeneous earthfill with a cutoff trench, downstream filter-drainage blanket and upstream riprap slope protection. The reservoir which is approximately 1.2 km long and 0.7 km wide drains a watershed basin of approximately 60 square km.

The spillway system includes a CSP drop inlet operating or service spillway and an auxiliary earth cut channel spillway.

A riparian outlet which is a requirement of the water licence consists of a concrete inlet, a 200 mm PVC pipeline, a buried gate valve and a riprap plunge basin.

The Project Summary Sheet lists pertinent data associated with the project.

1.4 Project Record

All the activities specified in this manual associated with operation, maintenance and inspections are to be recorded in a Project Log maintained by the owner. An example of a project Log is included in this section.
John Doe Dam Project Data Summary Sheet

I. GENERAL

Project Name: John Doe Dam

Owner: John Doe

Province: Saskatchewan

Location: SW1/4-6-26-16-3

Stream: Sometimes Dry Creek

Gross Drainage Area: 60 Km²

Construction: 1988

Hazard Potential Classification: low (CCC)

Project Inflow Design Flood: 1:100

Project Components: Embankment, Operating Spillway, Emergency Earth Spillway, and Riparian Outlet.

Use: Irrigation

Full Supply Level (FSL): 101.0 m

Storage Capacity at FSL: 200 dam³

Storage Capacity at TOD: 340 dam³

Flooded Area at FSL: 135 ha

Maximum Flood Surcharge Elevation: 101.8 m

II. EMBANKMENT

Description: homogeneous, impervious earthfill.

Upstream Slope: 3:1

Downstream Slope: 2.5:1

Top of Dam elevation: 102.3 m

Crest Length: 70 m

Crest Width: 3.0 m

Maximum Height: 7.6 m

Freeboard at FSL: 1.3 m

III. (A) OPERATING SPILLWAY

Description: CSP drop inlet spillway

Riser Diameter: 1000 mm

Riser Crest Elevation: 101.0 m

Riser Length (Crest-Invert): 5.7 m

Conduit Diameter: 500 mm

Inlet Invert: 95.9 m

Outlet Invert: 95.8 m
John Dam Project Data Summary Sheet (Cont'd)

Conduit Length: 21.0 m
Design Surcharge Elevation: 101.3 m
Discharge at Design Surcharge: 0.8 m³/s
Discharge at Project Flood Surcharge Elevation (101.8 m): 0.9 m³/s

III. (B) AUXILIARY SPILLWAY

Description: earthcut channel
Inlet Invert: 101.3 m
channel Bed Width: 20 m
Channel Sideslopes: 3:1
Channel Bed Slope: 0.0000
Channel Length: 30 m
Flood Surcharge Elevation: 101.8 m
Discharge of Maximum Flood Surcharge Elevation (102.0 m): 4.3 m³/s

IV. RIPARIAN OUTLET

Description: PVC conduit complete with concrete inlet, buried gate valve and riprap plunge pool
Conduit Diameter: 200 mm
Gate type and size: 200 mm
Conduit Length: 32 m
Invert Elevation at Inlet: 95.2 m
Invert Elevation at Outlet: 95.1 m
Discharge at FSL: 0.12 m³/s
# Project Log

**PROJECT:** John Doe Project on Sometimesdry Creek

<table>
<thead>
<tr>
<th>DATE</th>
<th>OPERATOR</th>
<th>TYPE OF WORK</th>
<th>REMARKS</th>
</tr>
</thead>
<tbody>
<tr>
<td>April 15/89</td>
<td>Owner</td>
<td>Inspection</td>
<td>Reservoir elevation 101.2. Spring runoff occurring. Operation spillway ok.</td>
</tr>
<tr>
<td>May 17/89</td>
<td>Owner</td>
<td>Inspection</td>
<td>Minor erosion of slope protection on left abutment.</td>
</tr>
<tr>
<td>July 16/89</td>
<td>Owner</td>
<td>Maintenance</td>
<td>Place rocks in erosion area on left abutment.</td>
</tr>
<tr>
<td>July 30/89</td>
<td>Owner</td>
<td>Operation</td>
<td>Open riparian gate for flow of 0.1 m³/s. Reservoir elevation 100.7</td>
</tr>
<tr>
<td>Aug. 30/89</td>
<td>Owner</td>
<td>Operation</td>
<td>Close riparian gate. Reservoir elevation 98.0.</td>
</tr>
<tr>
<td>Sept. 30/89</td>
<td>PFRA Technician</td>
<td>Inspection</td>
<td>Project in satisfaction condition. (See reference checklist).</td>
</tr>
</tbody>
</table>
2.0 OPERATION

2.1 Normal Operation

Normal operation is utilization of stored water for irrigation and riparian demands.

(The owner determines his own demands and withdrawal schedule based on the type of crop, seasonal weather conditions and type of operation.) Pump rates, duration and reservoir levels are recorded to monitor performance of the reservoir.

(The riparian releases are dictated by the terms of the water licence.) The operator shall comply with the riparian requirements and record gate openings, duration, and reservoir water levels during riparian releases.

The reservoir FSL shall not be raised by modifying dam components or by constructing dikes or any other methods. Such operation changes the design conditions and increases the risk of damage due to failure. Such action may also jeopardize project approval and result in cancellation of the water licence.

2.2 Flood Operation

Flood operation occurs when the spillway system is functioning to pass floods. Reservoir withdrawals for irrigation or riparian demands may occur during the flood operation.

The components of the spillway system operate automatically and therefore require no adjustments before, during, or after the flood operation. The operator should maintain the inlets of both the operating and auxiliary earth spillways free from trash and debris at all times. Before spring runoff the operator shall clear and maintain at least a pilot channel through any snow blockages of the auxiliary spillway.

Flood operation shall be documented in the Project Log by recording reservoir levels and duration of flood operation.
3.0 MAINTENANCE

3.1 Preventative Maintenance

Preventative maintenance for dam projects usually results in relatively low cost repairs from normal wear and tear during the project life.

A schedule for preventative maintenance of various project components is enclosed in this manual. All maintenance is to be reported in the Project Log.

3.2 Special Emergency Maintenance

If a defect is discovered in any component or the dam is threatened due to unusual conditions, repairs may be required to retain the capability and integrity of the dam. Technical advice for the method and extent of repairs may be requested from PFRA.
## Preventative Maintenance Schedule

<table>
<thead>
<tr>
<th>PROJECT COMPONENT</th>
<th>PREVENTATIVE MAINTENANCE REQUIREMENTS</th>
<th>FREQUENCY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site</td>
<td>1. Clear access road prior to spring runoff</td>
<td>X</td>
</tr>
<tr>
<td>Embankment, Abutments, And</td>
<td>1. Cut down and remove shrubs and trees growing on embankment.</td>
<td>X</td>
</tr>
<tr>
<td>Foundation</td>
<td>2. Eliminate burrowing animals and backfill burrow.</td>
<td>X</td>
</tr>
<tr>
<td>Operating Spillway</td>
<td>1. Cut and dispose of vegetation growing in approach and exit channels.</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td>2. Remove rocks and other debris from riser and conduit.</td>
<td>X</td>
</tr>
<tr>
<td>Emergency Earth Spillway</td>
<td>1. Cut and burn vegetation growing in channel.</td>
<td>X</td>
</tr>
<tr>
<td>Riparian and Irrigation Outlets</td>
<td>1. Lubricate hinges on gatewell covers.</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td>2. Lubricate gate hoist mechanisms and gate control stems.</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td>3. Operate all gates to full open position.</td>
<td>X</td>
</tr>
<tr>
<td></td>
<td>4. Remove obstructions and other debris from exit channel.</td>
<td>X</td>
</tr>
</tbody>
</table>

**Notes**

1. Other maintenance requirements identified during routine semi-annual or special inspections should also be performed as required.

2. Key to scheduled frequency: M - monthly; SA - semi-annually; and A - annually.
4.0 INVESTIGATION

4.1 Routine Inspections

Frequent routine inspections provide an awareness or knowledge of the condition and performance of the project. This provides early identification of problem areas which can be rectified in the most efficient, cost-effective manner.

Whenever the operator is in the vicinity of the project, he should take the time to perform a routine inspection. Some of the items to be inspected are described in the enclosed checklist. The date of inspection and observations should be recorded in the Project Log.

4.2 Semi-Annual and Special Inspections

An inspection of all project components should be performed by the owner during the spring runoff at a time of high water levels and maximum discharges. Another inspection should be performed in the autumn during a period of low water levels. The enclosed checklist is also a useful reference for the semi-annual inspection. A table of various adverse or potentially serious conditions which may be identified during an inspection and their relative severity with respect to safety and maintenance is provided at the end of this section.

A special inspection should be performed by the owner after any severe or unusual event such as a heavy summer rainstorm or tornado.
Routine Inspection Checklist

1. SITE
   - condition of access road.

2. EMBANKMENT, ABUTMENTS, AND FOUNDATION
   - slope protection/erosion
   - cracks and slides
   - movement and settlement
   - seepage and wet areas
   - animal burrows.

3. OPERATING SPILLWAY
   a. Approach Channel
      - obstruction and vegetation.
   
   b. (1) Concrete Structure       (2) CSP Structures
      - cracks                     - leakage
      - movement                   - debris
      - leakage                    - corrosion.
   - debris.
   c. Exit Channel
      - obstructions and vegetation
      - channel protection.

4. EMERGENCY EARTH SPILLWAY
   - obstructions and vegetation
   - erosion.

5. RIPARIAN AND IRRIGATION OUTLETS
   a. Gates and Gatewell
      - leakage
      - hoist mechanism
      - security.
   
   b. Energy Dissipater
      - erosion
      - material deposits and seepage.
   
   c. Exit Channel
      - obstruction and vegetation
      - channel protection.
## Adverse Conditions for Dam Projects

<table>
<thead>
<tr>
<th>Condition Requiring Normal Preventative Maintenance</th>
<th>Condition Requiring Immediate Special Maintenance Based On Technical Advice</th>
<th>Evacuation of Conditions General Areas</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Vegetative growth restricting seepage or visibility.</td>
<td>1. Seepage, boggy areas or changes in historical seepage patterns downstream of embankment.</td>
<td>1. Excessive seepage through embankment with erosion of fines.</td>
</tr>
<tr>
<td>2. Leaking gates.</td>
<td>2. Minor overtopping of embankment.</td>
<td>2. Excessive overtopping of embankment.</td>
</tr>
<tr>
<td>3. Surface deterioration of construction material such as concrete, metalwork, or timber.</td>
<td>3. Open construction joints in concrete.</td>
<td></td>
</tr>
<tr>
<td>4. Erosion of protective covering such as RIPRAP or bedding gravel.</td>
<td>4. Cracks, severe settlement or slides in embankment or abutments.</td>
<td></td>
</tr>
<tr>
<td>5. Build-up of debris in entrance channel, exit stilling basin.</td>
<td>5. Erosion of earth spillways.</td>
<td></td>
</tr>
<tr>
<td>6. Inoperable gates or hoists.</td>
<td>7. Corroded conduits.</td>
<td></td>
</tr>
</tbody>
</table>

**Note:**

1/ Technical advice is available from PFRA.
List of Standard Drawings

SD01 Area - Storage Capacity, Example Calculation Using Cross-Section Data

SD02 Riprap Plunge-Pool Energy Dissipater

SD03A Earth Channel Spillway (width 1-10 m)
SD03B Earth Channel Spillway (width 10-1000 m)

SD04 CSP Drop Inlet Spillway

SD05 CSP Drop Inlet Spillway - Stoplog Riparian Structure

SD06 CSP Drop Inlet Spillway - Gate Valve Riparian Structure

SD07 Precast Concrete Pipe Drop Inlet Spillway

SD08 CSP Sloping Pipe Spillway

SD09 CSP Riparian Structure

SD10 PVC Riparian Structure - Type I

SD11 PVC Riparian Structure - Type II

SD12 Reinforced Concrete Stoplog Structure - Structure Layout

SD13 Reinforced Concrete stoplog Structure - Reinforcing Details

SD14 Rock-Armoured Overflow Spillway
APPENDIX A - STANDARD DRAWINGS

A-1 General

The drawings enclosed and discussed in this section were prepared by the professional engineering resources within PFRA based in part on standard engineering practice and in part on the experience derived from the monitoring of structures constructed in the past. It has been assumed for purposes of design that the expected project life is 15-20 years and the project owner-operator is prepared to accept some degree of risk for maintenance and repairs. The standard drawings in this appendix describe various structures commonly used in small dam projects. Special or unusual structures should be referred to a qualified professional engineer.

This appendix of standard drawings can be updated and extended. The drawings are in sufficient detail and form to prepare project component drawings for cost estimates, planning studies, project approvals, construction, and operation and maintenance manuals.

A standard drawing shows the structure elements, dimensions and material type in the form of specific details or design parameters.

Various design parameters (structure dimensions and thicknesses) are related to a flow discharge, dam height, or characteristic dimension such as a conduit diameter through tables or graphs on standard drawings.

By utilizing the information on the standard drawings and the data unique to each specific project (dam elevation, natural ground elevation, structure location) a project structure drawing is readily prepared from the "form" or blank standard drawings shown in the "Attachments" section of this manual.

Each of the standard drawings is described below with respect to function, advantages, and disadvantages.

A-2 Drawing SD01 Area - Storage Capacity, Example Calculation Using Cross-Section Data

Although this drawing is not a standard drawing in the usual sense, it was included as a reference or example of one manual method of computing storage volumes and flooded areas for reservoirs using cross-section information. Area storage capacity may also be obtained from topographical information. Alternative manual methods and computer programs are also available for these computations.

A-3 Drawings SD02 Riprap Plunge-Pool Energy Dissipater

A Riprap Plunge Pool consists of a rock-lined basin complete with bedding gravel, located under and downstream from a cantilevered conduit. The plunge pool dissipates the energy of pipe flows through turbulence.
This standard drawing contains design parameters for two design rock sizes (Design A and Design B) as a function of head and conduit diameter. The head is defined as the difference between FSL and conduit outlet invert elevation. Design A is based on a different design rock size (d50) for each conduit diameter considered. Design B is based on a constant design rock size (d50) of 300 mm. This standard drawing has no "form" or blank drawing for producing a project drawing. Information from this standard drawing is used or specified on the form drawings of other standard drawings to produce a pertinent project structure drawing.

A riprap plunge-pool energy dissipater provides a relatively economical method of controlling high velocity flows from pipe spillway and riparian outlets. It is anticipated that under normal operating conditions some minor maintenance will be required to ensure satisfactory performance.

Alternate energy dissipaters conventionally use reinforced concrete materials which normally provide low maintenance, are also relatively expensive, and are not considered in the manual.

A-4 Drawings SD03A and SD03B Earth Channel Spillway

These particular drawings although not for use specifically as a construction drawing was included in the standard drawing catalogue for convenience. These drawings are a design aid for locating and sizing an earth channel spillway. The details for an earth channel spillway for a particular project would be shown on the Project Location, Layout, and Details drawing prepared by the designer for each specific project.

Description
-----------

An earth channel spillway is an excavated trapezoidal channel adjacent to but at some distance from the embankment materials. The channel generally has a flat bed slope (0%) with the inlet invert at FSL or at the surcharge elevation of the operating spillway. The downstream end of the channel should exit to a natural draw or uncultivated area with a natural vegetation cover.

The earth channel spillway usually serves as a source of borrow material for construction of the dam embankment. It is usually sized (channel width) to pass the Safety of Dam Design Flood in conjunction with an operating spillway.

Advantages
--------

The earth channel spillway is a low cost, large capacity spillway. Since the excavated channel material is used in the embankment, the channel width is not sensitive to cost and large spillway capacity can be obtained while keeping flow depths to a practical minimum. The wide channel width and corresponding low flow depths, minimizes flood...
surcharge and the overall embankment height. Low flow depths also minimize erosion in the channel section discharge point.

Disadvantages
-------------

As the earth channel spillway is constructed of soil materials, it is susceptible to erosion. In order to minimize high erosive channel velocities, the channel bed slopes are flat; and the channel terminates in an area protected with natural vegetative cover away from the embankment. The excavated channel should be seeded to grass for protection against erosion. Operation of the earth channel spillway will result in erosion. The erosion is normally most pronounced where the flat bottomed channel exits into the coulee or onto uncultivated land. With successive operation of the earth spillway, the erosion forms a gully that moves upstream towards the reservoir. Over the project life maintenance consisting of replacement of eroded material may be required to prevent erosion of the channel into the reservoir.

A-5 Drawing SD04 CSP Drop Inlet Spillway

Description
-----------

The CSP Drop Inlet Spillway consists of a combination CSP-concrete riser including a reinforced-concrete base section, a CSP barrel, and a riprap plunge-pool energy dissipater. The structure has a relatively low capacity and is utilized as an operating spillway in conjunction with an auxiliary earth channel spillway.

Advantages
----------

The CSP Drop Inlet spillway is constructed with primarily non-eroding material and would normally require a minimum amount of maintenance through the project life. The CSP riser section is shop fabricated for convenient installation. The concrete forming is simple and the reinforcing steel placement is uncomplicated with straight bars cut to suit in the field. The structure operates automatically and requires no adjustment or control.

Disadvantages
-------------

The structure has a small capacity in relation to its cost and is normally used in conjunction with a larger capacity auxiliary spillway.
Description

The CSP Drop Inlet Spillway and Stoplog Riparian Structure consists of a concrete inlet; a CSP upstream conduit; a CSP - concrete riser including interior stoplog facilities and a reinforced concrete base; a CSP downstream conduit; and a riprap plunge pool energy dissipater. Operation of the stoplog facility controls the discharge from the reservoir through the inlet and upstream conduit and regulates the elevation of the reservoir. During floods water flows over the stoplogs as the reservoir rises. When the reservoir exceeds FSL, flood waters pass over the drop inlet crest section.

This structure combines the function of reservoir outlet and a drop inlet spillway. The spillway function of the structure has a relatively small capacity and is used as an operating spillway in conjunction with a large capacity auxiliary spillway.

Advantages

The combined CSP Drop Inlet Spillway and Stoplog Riparian Structure is constructed primarily of non erodible materials and would normally require minimum maintenance throughout the project life. The riser section is shop fabricated for convenient field installation. The concrete forming is not complex. The steel reinforcing is uncomplicated and consists of straight bars cut to suit in the field. The spillway works automatically and requires no operation. The combination of two structures in one offer cost savings compared to two separate structures.

Disadvantages

The capacity of the structure when acting as a spillway is relatively small and must be used in conjunction with a large capacity auxiliary spillway. The stoplog facility is within the reservoir and may be inconvenient to operate at certain high water levels.

Leakage that may occur through the stoplogs results in wasted water to the downstream channel. Stoplog leakage in the autumn and winter can result in plugging of the outlet pipe due to ice formation.

Stoplogs in the drop inlet riser will result in marginally reduced capacity over that of a conventional CSP drop inlet spillway.
A-7 Drawing SD06 CSP Drop Inlet Spillway and Gate Valve Riparian Structure

Description
----------

This structure is similar to the CSP Drop Inlet Spillway Stoplog Riparian Structure. In this structure a gate valve housed adjacent to the vertical riser section replaces the stoplog system for controlling the reservoir level.

The combined CSP Drop Inlet spillway and Gated Riparian Structure consists of a concrete inlet; a CSP upstream conduit; a CSP - concrete riser including a CSP housing and gate valve and a reinforced-concrete base; a CSP downstream conduit; and a riprap plunge pool energy dissipater. Operation of the gate valve controls the discharge from the reservoir through the inlet and upstream conduit and regulates the elevation of the reservoir. When the reservoir exceeds FSL, flood waters pass over the drop inlet crest section.

This structure combines the function of reservoir outlet and a drop inlet spillway. The spillway function of the structure has a relatively small capacity and is used as an operating spillway in conjunction with a large-capacity auxiliary spillway.

Advantages
----------

The combined CSP Drop Inlet Spillway and Gated Riparian Structure is constructed primarily of non erodible materials and would normally require minimum maintenance throughout the project life. The riser section including gate valve and fittings are shop fabricated for convenient field installation. The concrete forming is not complex. The steel reinforcing is uncomplicated and consists of straight bars cut to suit in the field. The spillway works automatically and requires no operation. The combination of two structures in one offers cost savings compared to two separate structures. With correct operation of the gate valve, no leakage should occur when the valve is closed.

Disadvantages
----------

The capacity of the structure when acting as a spillway is relatively small and must be used in conjunction with a large capacity auxiliary spillway. The gate valve control is within the reservoir and may be inconvenient to operate at certain high water levels.

A-8 Drawings SD07 Precast Concrete Pipe Drop Inlet Spillway

Description
----------

This structure is similar to the CSP Drop Inlet Spillway, however, this standard drawing has only one conduit size, 600 mm.
The Precast Concrete Pipe Drop Inlet spillway consists of a precast concrete pipe riser including a reinforced-concrete base section; a CSP conduit; and a riprap plunge-pool energy dissipater. The structure has a relatively low capacity and utilized as an operating spillway in conjunction with an auxiliary earth channel spillway.

Advantages
-----------

The Precast Concrete Pipe Drop Inlet spillway is constructed with primarily non eroding material and would normally require a minimum amount of maintenance through the project life. The concrete forming is simple and the reinforcing steel placement is uncomplicated with straight bars cut to suit in the field. The structure operates automatically and requires no adjustment or control.

Disadvantages
-------------

The structure has a small capacity in relation to its cost and is normally used in conjunction with a larger capacity auxiliary spillway.

A-9  Drawing SD08 CSP Sloping Pipe Spillway

Description
-----------

The CSP Sloping Pipe Spillway consists of a CSP conduit including a fabricated steel hooded inlet, and a riprap plunge-pool energy dissipater. The structure has a relatively low capacity and utilized as an operating spillway in conjunction with an auxiliary earth channel spillway.

Advantages
-----------

The CSP Sloping Pipe Spillway is constructed with primarily non eroding material and would normally require a minimum amount of maintenance through the project life. No concrete forming is required for the construction of this structure. The hooded inlet section is shop fabricated for convenient installation. The structure operates automatically and requires no adjustment or control.

Disadvantages
-------------

The structure has a relatively small capacity and is normally used in conjunction with a larger capacity auxiliary spillway. The structure requires a relatively large reservoir surcharge to provide the design discharge. This has the effect of increasing the freeboard allowance and the top of dam elevation.
The CSP Riparian Structure consists of a concrete inlet structure, a CSP upstream conduit, a fabricated CSP gatewell complete with concrete base, a downstream CSP conduit, and riprap-armoured length of exit channel. The structure provides relatively high discharges to be independently released from the reservoir.

Advantages
----------

This structure is constructed from readily available conventional materials (steel, concrete, rock) and will provide reliable low-maintenance service through the project life. The concrete inlet structure provides anchorage and protection to the pipe inlet, and prevents trash from entering and plugging or damaging the structure. This is a relatively low-cost structure for providing medium to large releases from a storage reservoir.

Disadvantages
-------------

This type of structure which utilizes a slide gate is prone to leakage which may waste water and cause ice conditions in the downstream conduit and exit channel. Under some conditions deterioration of the CSP materials may occur through corrosion. In these cases and in situations where long project life is a requirement non-corrosive materials should be used or the CSP material protected with a corrosion protection system.

A-11 Drawings SD10 PVC Riparian Outlet Structure - Type I and SD11 PVC Riparian Structure - Type II

Description
-----------

These two structures are basically the same except for different pipe inlet and outlet treatments. The inlet of the Type I structure comprises a length of CSP including a trash rack inserted over the upstream PVC pipe. The Type II structure includes a concrete inlet complete with trash rack. The outlet of the Type I structure comprises a section of CSP inserted over the end of the downstream section of the PVC pipe. The Type II structure has no outlet pipe treatment.

Both types of riparian PVC structures consists of a PVC pipe through the embankment with an inlet structure, a buried gate valve near the downstream end of the pipe and a riprap armoured section of exit channel. These structures provide relatively small-capacity, controlled gravity releases downstream from the reservoir for riparian or other scheduled use.
Advantages
----------

As the pipe material is plastic, the structure is not susceptible to corrosion. The reinforced-concrete inlet provides anchorage, and protection to the pipe inlet. The trash racks of both structure types prevents trash from entering and plugging or damaging the structure. The CSP inlet and outlet treatments may provide protection to the PVC pipe from ice conditions; ultra-violet rays of sun, and impacts of normal wear and tear. The gate valve is buried for frost protection, however, it is still practically accessible using small equipment for maintenance and repair. A downstream berm provides a margin of safety to the embankment in the event of erosion or damage to the pipe at the outlet section.

Disadvantages
-------------

The use of plastic pipes is limited to relatively small pipe sizes, therefore, the pipe is not accessible for inspection. The pipe upstream of the gate valve is under pressure adjacent to the downstream end of embankment. Pipe leaks upstream of the valve may result in embankment erosion or instability because of the relatively short seepage path to the embankment toe. Repair and maintenance of the valve would require draining the reservoir or isolating the reservoir from the inlet by means of a diver placed plug or cofferdams.

Description
-----------

The Reinforced Concrete Stoplog Structure consists of a horizontal reinforced-concrete rectangular section through a dyke or dam embankment complete with stoplog facilities. Operation of the stoplog facility controls the reservoir elevation and discharge through the structure. This type of structure is normally used to provide continuous discharge from a reservoir or stream.

Advantages
----------

The Reinforced-Concrete Structure is constructed of non erodible materials and would normally require minimum maintenance throughout the project life. The concrete forming is not too complex. The reinforcing steel was laid out with a minimum of bent bar shapes and bar sizes. Straight bars are cut to suit in the field.

Disadvantages
-------------

This structure is a relatively low-head structure as the structural capacity of the stoplogs limits structure height and width.
**Description**

The Rock-Armoured Overflow Spillway consists of a prepared graded excavation covered with a layer of bedding gravel and riprap to form a chute-type overflow spillway. A steel sheet pipe cutoff system controls seepage and provides a fixed, uniform weir crest elevation.

**Advantages**

This type of structure provides relatively moderate flows with a low head or reservoir surcharge.

**Disadvantages**

This structure is constructed primarily of rock materials and is subject to erosion. Some degree of maintenance may be required over the life of this structure.
CALCULATION OF SURFACE AREA AT FSL:

Surface Area A = \( \frac{(87.0+48.7)}{2} \times 70.1 = 4055.3 \text{ m}^2 \)

Surface Area B = \( \frac{(48.7+24.4)}{2} \times 54.9 = 2006.6 \text{ m}^2 \)

Surface Area C = \( \frac{1 \times 24.4 \times 42.7}{2} = 520.9 \text{ m}^2 \)

TOTAL = 6582.8 \text{ m}^2

\( \ast \), Total Reservoir Surface Area = 0.7 ha.

CALCULATION OF STORAGE CAPACITY AT FSL:

Area of Sec 2 = \( \frac{1}{2} \times 0.9 \times 12.2 \times \left[ \frac{(0.9+1.8)}{2} \times 9.1 \times \left[ \frac{(1.8+1.0)}{2} \times 9.1 \right] \times \frac{1}{2} \times 1.8 \times 18.3 \right] = 108.8 \text{ m}^2 \)

Area of Sec 3 = \( \frac{1}{2} \times 0.9 \times 6.1 \times \left[ \frac{(0.9+1.8)}{2} \times 9.1 \right] \times \left[ \frac{(1.8+1.0)}{2} \times 15.2 \right] \times \frac{1}{12} \times 0.3 \times 9.1 \] = 46.0 \text{ m}^2

Area of Sec 4 = \( \frac{1}{2} \times 0.9 \times 6.1 \times \left[ \frac{(0.9+1.8)}{2} \times 9.1 \right] \times \left[ \frac{(1.8+1.0)}{2} \times 3.1 \right] \times \frac{1}{2} \times 0.4 \times 6.1 \] = 10.5 \text{ m}^2

Storage Capacity A = \( (108.8 + 46.0) \times 70.1 = 5355.6 \text{ m}^3 \)

Storage Capacity B = \( \frac{(46.0 + 10.5)}{2} \times 54.9 = 1550.9 \text{ m}^3 \)

\( \ast \), Storage Capacity C = \( \frac{1}{3} \times 10.5 \times 42.7 = 149.5 \text{ m}^3 \)

TOTAL = 7056.0 \text{ m}^3

\( \ast \), Total Reservoir Storage Capacity = 7.06 \text{ dam}^3

Note: (1) Reservoir divided into reaches A, B, and C.
(2) Simple dimensions have been chosen for illustrative purposes only.
(3) Dimensions shown in metres.

Storage capacity for reservoir reach C is calculated using the prismoidal volume formula rather than by average and area methods. This results in a conservative estimate of the reservoir volume.
This drawing is a design aid only and not for construction.

H = difference between water level in reservoir and invert elevation of earth spillway.

LEGEND
Q = spillway design discharge in m³/s
L = length of spillway in metres
H = head on inlet (in metres)
W = bottom width of spillway in metres
n = Manning's coefficient

PROCEDURE
Knowing L, H and Q, determine W from curves provided.

EXAMPLE
Know Q = 2.5 m³/s (from runoff curves)
Know L = 100 m (from topography)
Select H = 0.50 m (design variable)
Enter curves with L = 100 m

H = 0.50 m and Q = 2.5 m³/s
Then W = 8 m.
APPENDIX B

SPECIFICATIONS FOR CONSTRUCTION
List of Specifications

Specification for Earthwork
Specification for Corrugated Steel Pipe (CSP)
Specification for Reinforced Concrete
Specification for Miscellaneous Metalwork
Specification for Gate Valves
Specification for Polyvinyl Chloride (PVC) Pipe
Specification for Precast Concrete Manholes
Specification for Slide Gates
Specification for Polyethylene (PE) Pipe
General

Engineering designs are based on assumptions regarding the quality of materials and workmanship used in the project construction, and operation and maintenance during the project life. The specifications assist in describing and setting out the expected quality for construction and the assumed utilization of the project works. In order to ensure the safety of the project and structural integrity of the project components, conformance with the specifications for construction is necessary.

This section includes typical specifications for the various items shown in the standard drawings in Appendix A. These specifications may be copied and compiled with the standard drawings to produce a set of documents for project approval, tendering or construction.

These specifications have been compiled and abbreviated from various specifications used for large engineered dam projects designed by PFRA. The specifications are to be reviewed and updated as various material standards and construction methods change with time.

At the end of each specification is placed a measurement and payment clause. These are included for completeness. The contracting authority (project owner) may elect to administer the payment portion of a contract in a different manner from that stated in these clauses. In this case the measurement and payment clauses would be changed or deleted.

If the measurement and payment clauses are to be used for compensation for the work, then the contracting authority should establish with the Contractor, the prices that will be paid for each unit of work. The payment clauses in the specifications refer to a "Unit Price Table" which, if these clauses are to be used without modification, should be constructed. The table would include each item of work described in the specifications that is necessary for the construction of the project, and the price agreed between the Contractor and the contracting authority for performing that unit of work. For example, the agreed-upon price for "Excavation" as defined in the specifications could be $1.00 for each unit of work of 1 m³ of excavation. The total price paid for excavation would be determined by measuring the total volume of excavation (units) multiplied by price agreed upon for each unit. The same "Unit Price Table" could be used by the contracting authority to solicit prices or bids for the work for contractor selection purposes.
SPECIFICATION FOR EARTHWORK

Excavation

All excavation for the work shall be performed by the Contractor and shall include but not necessarily be limited to the following excavation operations:

1. Stripping shall include removal of vegetation, topsoil and all material unsuitable for forming foundations for the embankment or structures. Stripping shall also include removal of vegetation and topsoil from sources of borrow material and the earthcut spillway. Stripping material suitable for use as topsoil in grassing operations shall be stockpiled for later use at convenient locations so as not to interfere with the work. Stripping material not suitable for any purpose shall be wasted at locations which will not interfere with construction or operation of the dam.

2. Excavation for the key trench involves removal of soft, pervious, crumbly or fractured foundation soil to expose an impervious contact face. Excavations for structure involves removal of material to the desired elevation upon which to place the structure. Structure excavations shall not be made deeper than shown on the drawings unless specifically instructed to do so by the Owner. Materials suitable for use in the embankment shall be placed directly into the embankment or stockpiled for later use. Unsuitable materials shall be wasted.

If unsuitable material is exposed upon completion of the excavation for the foundation of a structure, the Contractor shall notify the Owner or inspector and shall not proceed with the construction of the structure until directed by the Owner or inspector.

3. Excavation for the earthcut spillway involves the removal of material to produce the desired cross-sectional shape at the specified elevation as shown on the drawings. Material suitable for use in the embankment shall be placed directly in the embankment or backfill areas or stockpiled for later use. Unsuitable materials shall be wasted.

4. Excavation in the borrow areas involves obtaining suitable material for use in the embankment or backfill from designated borrow areas and placing the suitable materials directly in the embankment or backfill. In cases where unsuitable material must be removed to expose suitable material, the unsuitable material shall be wasted.

Excavation includes loosening, digging, loading, hauling and disposal of excavated materials at the designated location or point of use. Excavations shall be planned to provide the most suitable materials for construction and to minimize the handling of material.
Permanent excavation slopes shall be constructed at 3:1 and maintained in a neat, uniform condition. Temporary excavation slopes shall not be steeper than permitted by the Worker's Compensation Board.

**Foundation Preparation**

The entire foundation surface shall be stripped of topsoil and organic matter. Crumbly or fractured foundation soil in the key trench shall be removed to the depth shown on the drawings, determined in the field, or specified in these specifications. If pervious material zones or soft material occurs below the stripped depth or crumbly, fractured soil occurs in the key trench below a depth of 2 m the owner or inspector should be notified and the Contractor shall stop foundation preparation operations. The owner or inspector will review the foundation preparation requirements in view of the existing field information and subsequently direct the Contractor on any additional requirements for foundation preparation.

The stripped foundation beneath the impervious section of the embankment, including the key trench, shall be thoroughly scarified to a depth of 150 mm by discing. This surface shall then be compacted to a density equivalent to that of the embankment. The foundation beneath the internal filter does not require scarification or compaction.

**Materials For Embankment Construction**

Embankment materials, impervious Zone 1 and random Zone 2 shall consist of low to medium plastic clay or a suitable homogeneous mixture of sand, gravel, silt or clay. This material should contain sufficient silt and clay fines to be relatively impermeable and suitable for compacting as specified. A zone downstream of an inclined or vertical filter may be constructed of random Zone 2 material as obtained from borrow sources without extra mixing other than that provided by standard excavating and spreading procedures. Embankment materials shall be free from vegetal growth, organic substances, stones greater than 150 mm diameter, ice, snow, and frozen materials.

Select filter sand Zone 3A material shall consist of clean, free draining, reasonably well-graded sand or sand and gravel mixtures having a maximum size of 75 mm and less than 4% silt or clay fines when in place.

Riprap bedding gravel Zone 3B shall consist of reasonably well-graded sand and gravel with at least 15% coarser than 38 mm and less than 8% silt or clay fines when in place.

Riprap Zone 4 and/or Zone 5 shall consist of hard, dense, durable fieldstone, cobbles or rock fragments but not sandstone, clay shale, or clay ironstone. Flat slabby type rocks shall be excluded. Riprap shall be reasonably clean. Riprap shall have an average size (D50) and thickness as shown on the drawings. Riprap shall be reasonably well-graded with no sizes lacking and no excess of material in any size range.
Moisture Requirements

Impervious Zone 1 materials shall have a water content between 1% dry and 3% wet of optimum water content to obtain maximum density upon compaction. Random Zone 2 material shall have a water content ranging from 2% dry to 4% wet of optimum water content. Material with water contents between optimum and 4% wet of optimum water content shall be used along conduits, at the abutment contacts and adjacent to concrete structures to ensure intimate contact between soil and structure.

The water content of the fill material can be increased by sprinkling water on an uncompacted loose lift with a water truck or by irrigating the borrow pit. If sprinkling a loose lift with a water truck is employed, the lift shall then be thoroughly mixed by discing before compacting. Processing material on the fill may be difficult due to a relatively small working area. The difficulty increases when a large increase in water content is required. These difficulties may be avoided and construction expedited by adding water in advance at the borrow pit.

Ponding or irrigation with sprinklers in the borrow pit should begin at least two weeks prior to beginning construction and preferably longer as the depth of moisture penetration is dependent on the length of soaking time and soil type. When sufficient penetration has not been achieved prior to construction, the borrow pit could be divided into three sections of more or less equal area with one of these sections always in the process of being wetted while the second is soaking and the third is being excavated. Drying of overly wet soils can be accomplished by discing the material and leaving it exposed to the atmosphere.

The water content of impervious materials in the borrow areas may change seasonally and should be confirmed just before construction. Allowance should be made for evaporation losses during excavating, haul- ing and placing. Losses in water content from evaporation are commonly in the range of 1% to 3% and may be as high as 5% or 6% depending on the weather.

Placing Procedures

Fill material shall be spread in lifts parallel to the embankment centreline to prevent a continuous stratified layer extending from upstream to downstream. The lift thickness, before and after compaction, shall not exceed 250 mm and 150 mm respectively.

Fill materials shall not be placed in a frozen condition or on a frozen surface. Winter construction should be avoided as the density obtained with a given compactive effort on impervious soil decreases noticeably when the temperature approaches the freezing point. However, granular materials may be placed in freezing conditions if the material is not placed in frozen lumps.
Granular materials may be placed in lifts of up to 300 mm thickness. Care shall be taken to prevent contamination of filter materials with clay and silt fines. The filter material may be end dumped from trucks at convenient locations and spread with a dozer or track loader. Care shall be taken to ensure that the filter exit is not covered with impervious material so that drainage can take place.

Riprap and bedding gravel may be placed by end-dumping from trucks and pushing the material into place. To avoid segregation which may occur with this method, tractors or winches may be used to lower trucks down the slope so that the rock can be dumped against previously placed rock. Alternatively, a front-end loader may be used from the bottom to travel over previously placed rock. In all placing methods some hand labour will be necessary to adjust surface rocks and to fill voids with smaller rock to produce a dense cover. Care shall be taken to insure that the specified thickness of riprap bedding exists under all riprap and that riprap placing methods do not displace the bedding gravel.

**Compaction Requirements**

Impervious fill shall be compacted to at least 95% of Standard Proctor maximum dry density. This can be achieved with 10 complete passes of a sheepsfoot roller having a unit foot pressure of at least 1700 kPa (250 psi) and preferably 2400 kPa (350 psi). The number of passes may be reduced to a minimum of 6 for random Zone 2 material. The foot length of the roller shall be at least 200 mm and the face area shall not be less than 4500 mm². The foot spacing shall provide approximately four (4) tamping feet for each one-quarter square metre of cylindrical surface.

Alternate compaction equipment may be used providing each compacted lift is scarified to a depth of at least 50 mm to prevent layering and at least 95% of Standard Proctor maximum dry density is obtained. Effort must be made to avoid uneven compaction and layering between lifts which may lead to subsequent problems of embankment settlement and seepage through the embankment to the downstream slope.

Compaction around conduits is critical since the conduit extends from the reservoir through the embankment to the downstream slope and seepage problems can easily develop. Backfill material shall be mechanically hand tamped within 900 mm of all structure and special care shall be taken to insure good compaction of all backfill beneath the haunches of circular conduits. Backfilling operations shall be conducted such that the fill is brought up evenly on each side of the structure or pipe to prevent unbalanced load on the structure. Equipment trackage can be utilized adjacent to conduits or concrete structures outside of the 900 mm zone; however, caution must still be exercised to ensure that the conduit or concrete structure is not damaged or pushed laterally. Select filter sand Zone 3A around the conduit and in the conduit foundation shall be compacted with vibratory equipment.
Riprap bedding gravel may be spread on the upstream face of the embankment with a dozer or track loader and does not require additional compaction.

Measurement

"Excavation" will be measured for payment by the cubic metre determined by either:

(a) Surveys made prior to commencement of the excavation and the lines and grades shown on the drawings or

(b) where no lines or grades are shown, surveys of the respective areas made prior to commencement of the excavation and after completion of the excavation.

"Compacted Embankment Zone 1 and/or Zone 2, Select Filter Sand Zone 3A, Bedding Gravel Zone 3B, Gravel Slope Protection Zone 3C, and Riprap Zone 4 and Zone 5" will each be measured for payment by the cubic metre determined by computation of the volume between (1) the foundation lines and grades where shown on the drawings or as determined on the basis of surveys made after embankment foundation excavation (stripping and key trench) except scarifying, and (2) the lines and grades of the completed structure as shown on the drawings or as modified by the contract authority.

Payment

"Excavation" including stripping, key trench and structure excavation, earth cut channel spillway excavation and borrow excavation will be paid for at the unit price per cubic metre bid therefor in the Unit Price Table. The bid price shall include the costs of excavating; loading, hauling, depositing in embankments, backfills, stockpiles or waste areas; and trimming and shaping of excavation, stockpile and waste areas.

"Compacted Embankment Zone 1A and/or Zone 2" will each be paid for at the applicable unit price per cubic metre bid therefor in the Unit Price Table. Such payments shall constitute full compensation for all work in connection with scarifying foundation and the spreading, drying if required, additional moisture if required, mixing, compacting, removing unsuitable material, rehandling embankment materials if necessary and all other incidental work required for the construction and protection of the embankment as specified. Payment will be made for this work regardless of the source of the placed materials, and will be in addition to any payment for excavating and transporting the material from necessary excavations or borrow areas.

"Select Filter Gravel Zone 3A, Bedding Gravel Zone 3B, Gravel Slope Protection Zone 3C and Riprap Zone 4 and Zone 5" will each be paid for at the applicable unit price per cubic metre bid therefor in the Unit Price Table. Such payments shall constitute full compensation for all work in connection with obtaining, processing, loading, hauling,
depositing at point of use, spreading, compacting if required, and all other incidental work required for completion of this work as specified.
SPECIFICATION FOR CORRUGATED STEEL PIPE (CSP)

General

The work covered by this section includes the supply of all materials, labour, plant and equipment for the supply, fabrication and installation of all corrugated steel pipe and corrugated steel pipe products as specified or shown on the drawings.

Materials

Corrugated steel pipe materials shall conform to CSA CAN3-G401-M81. The corrugated steel pipe shall be galvanized and shall have a corrugation profile of 68 mm pitch, 13 mm depth for pipe sizes up to and including 1200 mm, and a corrugation profile of 125 mm pitch, 26 mm depth for pipe sizes greater than 1200 mm. The material sheet thickness for various pipe sizes shall be as shown on the drawings.

Helical pipe ends shall be re-corrugated to provide a minimum of two annular corrugations for coupling purposes.

Couplings shall be used on all field connections and consist of galvanized materials of the same thickness as the pipes to be connected. The couplings shall be Hugger Band H-13 (semi-corrugated bands) complete with "O-ring" gaskets (2 gaskets per coupler) as supplied by Armco or approved equal. Each coupling shall consist of a one piece or two piece coupler complete with attached brackets, bolts and nuts. In addition, two 13 mm diameter galvanized rods with silo-type lugs shall be included as part of the coupler. The 13 mm diameter rods shall be fitted into the corrugations of the coupler and when drawn together provide increased joint strength.

Fabrication

The fabrication of CSP products shall conform to CSA CAN3-G401-M81. Fabrication shall be performed in the shop and the finished product transported to the field.

Finished members shall be straight and true, and have no protrusions or irregularities at fabricated elbow, tee or stub sections.

All surfaces of welds and areas where the galvanized coating has been damaged during welding shall be cleaned by wire brushing by hand or power tools. The cleaned surfaces shall be resurfaced with Devcon 2 or other approved protective coating conforming to CSA Standard CAN3-G401-M81. The Contractor shall contact the pipe supplier to determine the source and availability of repair coating.

Installation

Corrugated steel pipe products including pipes and fabricated items shall be placed according to the lines and grades shown on the drawings or as directed by the inspector or the owner. Installation procedures
shall be according to these specifications and the manufacturer's recommendations. If the pipe is damaged during installation, the Contractor shall repair or replace damaged pipe sections to the satisfaction of the owner.

The foundation for a pipe shall consist of a uniformly graded excavated bed free from soft or vegetal material, rocks, and lumps of dried soil materials. Pipes shall not be placed on fill materials except in the area of the internal granular filter system and key trench. Compacted select filter sand (Zone 3A) and impervious compacted embankment (Zone 1) shall form the pipe foundation at these locations, respectively. If foundations for structures or pipes are not suitable for placement of the structure at the elevation specified, the Owner or inspector will determine the corrective action necessary for the work to proceed.

The pipe lengths shall be placed so that the interior seam or lap is on the downstream side of a corrugated sheet. In coupling pipe joints, the "O-ring" gaskets shall be placed on each pipe end at the first valley of the annular corrugation; the coupling band shall then be placed around the joint and the pipes positioned such that the corrugations of the coupler lineup with the second annular corrugation at each pipe end; the coupling shall then be drawn together with the attached brackets and bolts. The 13 mm diameter rods with silo type lugs shall be positioned on each corrugation of the coupler and drawn together; finally the bolts through the brackets attached to the coupler shall be re-tightened. The complete length of pipe conduit shall be placed on the excavated foundation, coupled, inspected and approved by the inspector or owner before commencement of backfilling.

The backfill material shall be compacted embankment and/or select filter sand as specified in the standard specification for "Earthwork". The backfill material shall be brought up evenly on both sides of the pipe to the level of the pipe crown along the entire conduit length. Care shall be taken to ensure that backfill under the pipe haunches is thoroughly tamped into intimate contact along the full corrugated surface of the exterior pipe wall. The backfill shall be compacted to achieve a density equivalent to that obtained in the embankment using mechanical hand tamping equipment to at least 300 mm above the crown of the pipe. Above this level, backfilling may be performed by machine, however, the material shall be rolled, not dropped into any excavation.

CSP fabricated items shall be adequately supported during all operations including coupling, backfilling and concreting.

Measurement

Corrugated steel pipe (straight pipe) will be measured for payment by the lineal meter in place from end to end along the invert of each segment of pipe but not including fabricated items. Corrugated steel pipe fabricated items will be considered as lump sum items.
Payment

Corrugated steel pipe sections and fabricated items will each be paid for at the applicable unit price per lineal meter or lump sum bid therefore in the Unit Price Table. The price shall include the costs of supply, fabricating, handling, placing and assembling the pipes including connecting hardware, couplings, gaskets, lubricants, coatings and all other items required to make a proper installation as shown on the drawings and specified herein. The cost of earthwork including excavation and backfill is paid for under a separate section.
SPECIFICATION FOR REINFORCED CONCRETE

General

The work covered in this section consists of furnishing all plant, labour and equipment, and performing all work necessary for supplying, transporting, storing and handling of cement, aggregates, concrete admixtures, water and reinforcing steel; batching, mixing, transporting, forming, placing, finishing, curing and protecting the cast-in-place reinforced concrete required for the completion of this work.

Reinforced concrete work in place shall conform to required lines, elevations, dimension and details shown on the drawings.

Materials

All materials for concrete construction shall conform to CSA Standard CAN3-A23.1-M90 "Concrete Materials and Methods of Concrete Construction."

Concrete shall be proportioned and supplied to meet the following requirements:

1) Cement: Type 50, Sulphate-Resistant.
2) Specified 28-day compressive strength: 30 MPa.
3) Class of exposure: F1, Table 7 of CSA A23.1-M90.
4) Nominal maximum size of coarse aggregate: 20 mm.
5) Slump at point of discharge: 80 + 20 mm.
6) Air Content: 4 to 7 percent.
7) A type WN, normal setting water-reducing strength-increasing admixture may be used by the Contractor.
8) Supplementary cementing materials or other admixtures shall require prior approval by the owner or inspector.

Reinforcement shall be deformed bars of Grade 300 or higher billet steel conforming to the applicable provisions of CSA:G30.12-M77.

Methods

The methods for proportioning, producing, forming, reinforcing, placing, consolidating, finishing, protecting and curing concrete shall be in accordance with CSA:CAN3-A23.1-M90.

All concrete forms and forming shall be straight, true and plumb; complete; and securely anchored before placement of concrete. Before a new placement is begun adjacent to any horizontal, vertical or other construction joint, the surface of the hardened concrete shall
be cleaned of foreign matter and laitance. The surface cleaning shall be conducted in two phases. The first phase shall be performed approximately 10 to 15 hours after concrete is placed and shall entail scrubbing with wire brushes or wire brooms in order to remove laitance and expose the individual pebbles. The scrubbing shall be delayed for a few hours if the larger pebbles are loosened when the surface is scrubbed. The second phase of the surface cleanup shall be performed immediately before fresh concrete is placed on the hardened concrete. The surface shall be washed with water until the concrete is thoroughly cleaned of foreign matter and is thoroughly saturated. Directly before placing, free water shall be removed. The initial lift of concrete shall be not deeper than 300 mm and shall be thoroughly vibrated close to the joint to ensure complete contact of the mortar phase with the hardened concrete.

Sandblasting may be used in lieu of scrubbing for the first cleanup phase. This operation shall be performed on the hardened concrete prior to forming the next lift. Sandblasting shall be done with care and skill to remove the laitance film of mortar so that the coarse aggregate particles are exposed without undercutting the particles. After sandblasting, all loose particles shall be blown from the surface of the joint.

Concreting shall be performed in one continuous operation. Concrete shall be deposited in the forms as closely as practicable to its final position. Lateral movements of fresh concrete, which can cause segregation, shall be avoided. Concrete shall be placed in layers that are approximately horizontal. The depth of each layer shall be limited to ensure vibration into the previously placed layer. Concrete shall be confined in a suitable vertical drop pipe or trunk to within 1.5 m of the concrete in place.

Concrete when being placed shall be consolidated thoroughly and uniformly by means of hand tamping tools or vibrators to obtain a dense homogeneous structure free from cold joints, voids and honeycombs.

Concrete surfaces not bounded by forms shall be screeded and floated to provide a smooth dense surface finish. A special trowel or form shall be fabricated from sheet metal to provide radiused corners or edges as shown on the drawings.

Freshly deposited concrete shall be protected from premature drying and extremes of temperature to ensure proper curing of the concrete. Moist curing with water damp materials, curing compounds or plastic sheets shall be applied to concrete surfaces for a minimum of 7 days at an ambient temperature above 10° C.

**Measurement**

Concrete will be measured for payment by the cubic meter satisfactorily placed in the work, determined by computation of the volume of concrete shown on the drawings or as modified by the owner. No
deductions will be made for the volume of reinforcing steel and metal work embedded in the concrete.

**Payment**

Concrete will be paid for at the unit price per cubic meter bid therefore in the Unit Price Table. The price shall include the cost of furnishing all plant, labour, equipment, materials including cement and reinforcing steel required to complete the concrete work in accordance with the plans and specifications.

**Note**

It may be more efficient for contract administration purposes to have reinforced concrete bid on a lump sum basis rather than as a unit price item. If this is the case, then the measurement clause would be deleted and the appropriate changes would have to be made to the payment clause.
SPECIFICATION FOR MISCELLANEOUS METALWORK

General

The work covered in this section includes the supply of all materials, labour, plant and equipment for the supply, fabrication, and installation of all metalwork as specified or as shown on the drawings except corrugated steel pipes, reinforcing steel used in concrete and manufactured gates, valves, and accessories.

Materials

Metal materials shall be in accordance with the following table or as indicated on the drawings.

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
<th>Specification</th>
<th>Finish</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel Pipe Guardrails</td>
<td>Black, welded, Type F</td>
<td>ASTM:A53-82</td>
<td>Painted</td>
</tr>
<tr>
<td></td>
<td>standard weight</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>75 mm diameter</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Steel Pipe Conduit</td>
<td>Black, welded, Type F</td>
<td>ASTM:A53-82</td>
<td>Painted</td>
</tr>
<tr>
<td></td>
<td>standard weight</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Steel Chain</td>
<td>Class PC, 6 mm</td>
<td>ASTM:A413-80</td>
<td>Galvanized</td>
</tr>
<tr>
<td>Structural Steel</td>
<td>Plate, structural shapes</td>
<td>CSA:G40.21-M81</td>
<td>Painted</td>
</tr>
<tr>
<td></td>
<td>Bars and Rods</td>
<td>ASTM:A36M-87</td>
<td>Painted</td>
</tr>
<tr>
<td>Bolts, Washers</td>
<td>Hex Head</td>
<td>ASTM:A307-81</td>
<td>Galvanized</td>
</tr>
<tr>
<td>and Nuts</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Welding electrodes shall conform to the requirements of CSA:W48.1M1980.

Painting materials shall conform to the following Canadian Government Specifications Board (CGSB) and/or manufacturer's products number in accordance with the following:

(a) Primer - (CGSB:1-GP-48M "Primer, Marine, for Steel"

: #63048 Marine Primer by General Paint Ltd.

: #4570 Metal Primer by Glidden Paint Ltd.

(b) Marine Enamel Paint (CGSB:1-GP-61Ma "Enamel, Alkyd, Marine, Exterior and Interior")

: 10-010 Marine Enamel by General Paint Ltd.

: 22350 Super Marine Enamel by CIL Sherwin-Williams Paint
Steel shall be galvanized where shown on the drawings. Galvanizing shall be by the hot dip process in accordance with ASTM:A123-78.

**Fabrication**

All metal work shall be fabricated in sections in the shop and transported to the site.

Structural steel fabrication shall be in accordance with the applicable portions of CSA:CAN3-S16.1-M78, Steel Structures for Buildings - Limit States Design, clause 26.

Finished members shall be free from kinks, bends, or winds. Dimensions shall be accurate within 2 mm.

Welding shall be by the electric-arc-welding process using a method which excludes the atmosphere from the molten metal and shall conform to CSA;W59-84 and AWS:D2.0-1966.

Unless otherwise specified, all deposited weld metal shall have mechanical properties not less than those specified for base metal welded and shall have chemical composition similar to that of base metal.

All exposed ferrous surfaces not embedded in concrete shall be cleaned in the shop immediately after fabrication and prior to the application of the first coat of paint according to industry good practise and paint manufacturer's recommendations.

All exposed surfaces of welds executed in the field during the erection and mounting of the steel assemblies shall be cleaned in the field immediately prior to touch-up painting.

All metalwork to be painted shall receive one coat of primer and two coats of marine enamel paint.

Paint that is loose, weakly bonded, blistered, abraided or otherwise defective shall be removed and the surface shall be cleaned, prepared and repainted in accordance with the manufacturer's recommendations.

Painting shall not be performed during damp weather or when temperature is below freezing. Subsequent coats shall not be applied until the preceding coats are thoroughly dry.

**Installation**

All metalwork shall be correctly aligned and installed true to the lines and grades shown on the drawings and so that proper matching of adjacent concrete surfaces shall be obtained.

All anchors and anchor bolts shall be installed so that after placement of the concrete, their position is within 3 mm of their theoretical location and within 2 mm of their theoretical location with
respect to other anchors and anchor bolts which are to be subsequently connected to the same piece of metalwork.

All metalwork surfaces to be in contact with or embedded in concrete shall be thoroughly cleaned of all rust, dirt, grease, loose scale, grout, mortar or other foreign matter before installation.

Measurement

No measurement for Miscellaneous Metalwork is required.

Payment

Miscellaneous Metalwork will be paid for at the lump sum price bid therefore in the Unit Price Table. The price shall include all costs of furnishing, fabricating, galvanizing or painting where required and installing the materials as specified.
SPECIFICATION FOR GATE VALVES

General

The work covered in this section includes the supply of all materials, labour, plant and equipment for the supply and installation of gate valves as specified or shown on the drawings.

Materials

The gate valve shall be a Crane McAvity Resilient Seated Gate Valve complete with "flange" end and rubber gasket or approved equal.

The gate valve shall conform to ANSI/AWWA C509-80 "Resilient Seated Gate Valves, 3 through 12 NPS, for Water and Sewage System".

Miscellaneous valve stem extensions, support hardware, and fittings shall be as recommended by the manufacturer for buried service and as shown on the drawings.

Installation

Gate valves shall be installed according to the manufacturer's instructions.

Measurement

No measurement is required for Gate Valves.

Payment

Gate Valves will be paid for at the lump sum price bid therefore in the Unit Price Table. The price shall include all costs of furnishing and installing all materials as specified, including fasteners and hardware.
SPECIFICATION FOR POLYVINYL CHLORIDE (PVC) PIPE

General

The work covered in this section includes the supply of all materials, labour, plant and equipment for the supply and installation of PVC pipe as specified or shown on the drawings.

Materials

Polyvinyl Chloride (PVC) pipe shall be series 160, SDR 26 pipe as manufactured by IPEX or approved equal.

Polyvinyl Chloride (PVC) pipe described herein shall comply with the requirements of CSA/CAN3-B137.3-M86 "Rigid Poly (Vinyl Chloride) (PVC) Pipe for Pressure Applications".

The PVC pipe shall be homogeneous throughout, free from voids, cracks, inclusions and other defects, as uniform as practical in color, density and other physical properties. Surfaces of the products shall be free from scratches, gouges and other imperfections.

PVC pipe shall be coupled with factory molded bell ends or couplings with rubber ring joints. Flanged ends shall be installed at valve connections. PVC series pipe couplings and fittings shall be manufactured from CSA certified clean raw virgin material and the resin compound shall conform to ASTM:D1784-78, Class 12454-B and shall have a hydrostatic basis when tested and analysed by ASTM:D2837-76 of 27.58 MPa. Reworked material generated from the manufacturer's plant shall not be used to manufacture the pipe. The material shall be certified for potable water by the CSA Testing Laboratory.

Installation

Polyvinyl Chloride (PVC) pipe shall be installed to the lines and grades as shown on the drawings and according to the manufacturer's recommendation. The pipe shall be fully inserted into the bell end to accommodate potential movements.

The pipe shall be installed so that the barrel of the pipe is evenly supported throughout its entire length. The pipe shall be supported by a smooth, compacted bedding layer free from sharp projections, large dirt clods, stones greater than 40 mm in diameter or any frozen material. The backfill material shall be compacted in 100 mm layers to the specified density of the compacted embankment for the dam using mechanical hand tamping equipment to at least 300 mm above the top of the pipe to prevent undue pressures on the pipe. Above this zone, backfilling may be done by machines, however, the earth shall be rolled, not dropped into the excavations. All backfill shall be dry unfrozen material.

During all stages of construction, piping shall be protected from damage from any cause. Openings in the piping system shall be securely
covered, capped or plugged to prevent collection of dirt, debris, or other extraneous matter during the entire construction. Damaged work shall be removed and replaced with new material.

Before the pipeline is backfilled it shall be pressure tested according to the manufacturers recommendations.

**Measurement**

Polyvinyl Chloride (PVC) Pipe will be measured for payment by the lineal meter in place measured horizontally from point of origin to end of run with no deductions for adaptors, valves, or joints.

**Payment**

Polyvinyl Chloride (PVC) Pipe will be paid for at the unit price per lineal meter bid therefore in the Unit Price Table. The price shall include all costs of supply, hauling, handling, placing, assembling and testing the pipe including jointing materials and testing equipment. The cost of earthwork including excavation and backfill is paid for under a separate section.
SPECIFICATION FOR PRECAST CONCRETE MANHOLES

General

The work covered in this section includes the supply of all materials, labour, and equipment for the supply and installation of all precast concrete manhole risers as specified or shown on the drawings.

Materials

Precast concrete manholes shall be manufactured using Type 50 cement.

Precast concrete manhole risers shall conform to ASTM:C478M-85, "Standard Specification for Precast Reinforced Concrete Manhole Sections".

Access steps or ladder rungs are not required for manhole riser sections. The individual section lengths shall be determined based on the information shown on the drawings. The manhole supplier shall review the drawings and supply manhole section lengths to suit the project requirements. The manhole riser sections shall have tongue and groove ends. Manhole joints shall be made watertight using an O-ring rubber gasket conforming to ASTM C443M-85.

The bottom manhole section shall have an opening approximately 630 mm in diameter to receive a 600 CSP stub section as shown on the drawings. The opening shall be refinished with suitable grout with a minimum inside wall radius of 50 mm.

The galvanized metal material used to connect manhole riser section shall conform to CSA:G40.21-M81 or ASTM:A36M-87 for type of steel; ASTM:A307-81 for bolts, nuts and washers; and ASTM:A123-78 for galvanizing.

Installation

Manhole riser sections, rubber gaskets and lubricant shall be handled, stored, and placed according to the manufacturer's recommendations so as to prevent damage to the manhole sections and ensure a watertight joint. Special care shall be taken to ensure that joints and rubber gaskets are kept clean.

All manhole sections shall be placed with the spigot end pointing down and all joints connected with metal straps as shown on the drawings.

The complete length of manhole riser sections CSP conduit and concrete anchor and collars shall be placed, inspected and approved by the inspector or owner before commencement of backfilling.

The backfill material shall be compacted embankment (Zone 1) as specified in the standard specification for "Earthwork". The backfill material shall be brought up evenly on both sides of the riser section. The backfill shall be compacted to achieve a density equivalent to
that obtained in the embankment. Mechanical hand tamping equipment shall be used within a 1 m distance from the riser section.

**Measurement**

Manhole risers including fabricated pipe opening will be considered as a lump sum item.

**Payment**

Precast reinforced-concrete manhole risers will each be paid for at the applicable unit price per lump sum bid therefore in the Unit Price Table. The price shall include the costs of supply, fabricating, handling, placing and assembling the pipes including connecting hardware, couplings, gaskets, lubricants, and all other items required to make a proper installation as shown on the drawings and specified herein. The cost of earthwork including excavation and backfill is paid for under a separate section.
SPECIFICATION FOR SLIDE GATES

General

The work covered in this section includes the supply of all materials, labour, plant, and equipment for the supply and installation of slide gates as specified or shown on the drawings.

Materials

The slide gate shall be Armco Slide Gate Model 101C for the CSP pipe diameter shown on the drawings or approved equal.

The slide gate shall include a spigot back seat suitable for attachment to helical corrugated steel pipe with a maximum height angle frame. The gate shall also include a non-projecting stem extension assembly to suit the required stem height complete with operating nuts, pipe stem, sockets, guide collars and support hardware. The handwheel shall be type H1-10.

Installation

The slide gates shall be installed according to the manufacturer's instructions and as shown on the drawings. The intermediate pipe frame extension collar support shall be bolted to the pipe extension support bracket.

Measurement

No measurement is required for slide gate.

Payment

Slide gate will be paid for at the lump sum price bid therefor in the Unit Price Table. The price shall include all costs of furnishing and installing all materials as specified including fasteners and support hardware.
SPECIFICATION FOR POLYETHYLENE PIPE (PE)

General

The work covered in this section includes the supply of all materials, labour, plant and equipment for the supply, unloading, storage and installation of polyethylene pipe as specified or shown on the drawings.

Materials

Polyethylene pipe shall be series 100 - Sclairpipe as manufactured by Du Pont, Canada, Inc., or approved equal.

Polyethylene (PE) pipe shall conform to the requirements of CGSB:41-GP-25M, ASTM:D1248-81a, and ASTM:D2837-76(1981). The pipe shall be homogeneous throughout and free from cracks, holes, foreign inclusions, or other defects.

Fittings and high density PE pipe shall be manufactured from CSA certified clean raw virgin material. Materials shall conform to the requirements for Type III, Class C, Category 5, Grade P34, as per ASTM:D1248-81a, and have a hydrostatic design basis when tested and analyzed by ASTM:D2837-76(1981) of 10.0 MPa, as required by CGSB:41-GP25M.

The pipe shall be coupled by means of thermal socket or butt fusion, as per manufacturers specifications.

Fittings shall be manufactured from the same resin as the pipe and have dimensions suitable for fastening to the pipe by thermal fusion. Flanged fittings using cast aluminum slip-on metal back-up rings and polyethylene stub ends thermally fused to the pipe are acceptable. Stub end gaskets shall be manufactured from cloth-inserted rubber or asbestos fibre to the dimensions recommended by the pipe manufacturer. All nuts, bolts and washer hardware shall be as recommended by the manufacturer for buried service and as shown on the drawings.

Installation

Polyethylene pipe shall be installed to the lines and grades as shown on the drawings.

The pipe shall be installed so that the barrel of the pipe is evenly supported throughout its entire length. The pipe shall be supported by a smooth, compacted bedding layer free from sharp projections, large dirt clods, stones greater than 40 mm in diameter or any frozen material. The backfill material shall be compacted in 100 mm layers to the specified density of the compacted embankment for the dam using mechanical hand tamping equipment to at least 300 mm above the top of the pipe to prevent undue pressures on the pipe. Above this zone, backfilling may be done by machines, however, the earth shall be rolled,
not dropped into the excavations. All backfill shall be unfrozen material.

During all stages of construction, piping shall be protected from damage from any cause. Openings in the piping system shall be securely covered, capped or plugged to prevent collection of dirt, debris, or other extraneous matter during the entire construction. Damaged work shall be removed and replaced with new material.

Before the pipeline is backfilled it shall be pressure tested according to the manufacturers recommendations.

Measurement

Polyethylene pipe will be measured for payment by the lineal metre in place measured horizontally from point of origin to end of run with no deductions for adaptors, valves, or joints.

Payment

Polyethylene pipe will be paid for at the unit price per lineal metre bid therefore in the Unit Price Table. The price shall include all costs of supply, hauling, handling, placing, assembling, and testing the pipe including jointing materials and testing equipment. The cost of earthwork including excavation and backfill is paid for under a separate section.
APPENDIX C

COST ESTIMATING
C-1 General

Estimating the cost of project components or items and the overall project is part of the design process. The most economical alternative, component or detail can be determined or selected by cost comparison. The desirability or economical feasibility of a project can be assessed by comparing the cost of a project with potential benefits. Financial budgets and funding arrangements or agreements can be arranged using estimates of project costs.

It is the responsibility of the designer to see that representative cost estimates are provided during design optimization and for project assessment. Cost estimates must take into account local site conditions, availability and price of materials and labour, and seasonal conditions.

This section provides a guideline for selecting items of work within a project which are constructed as a unit and can be conveniently costed from information provided by material suppliers and contractors or cost information from previous projects.

C-2 Cost Items

The project costs can be considered as a series of cost items associated with either the embankment or structures. Embankment costs can be separated into the following items:

(a) Excavation - includes supplying the materials, labour, and equipment for excavating, loading, depositing, and trimming of slopes at designated places including the stripping (removal of vegetal material) of the embankment foundation, the auxiliary earth spillway, the inlet and outlet channels for the reservoir outlet structure, and any other required areas. It is normally paid for by the volume in cubic metres as determined from the drawings or surveys before and after the work is performed.

(b) Haul - includes supplying the labour and equipment for transporting the loaded excavation material from the place of excavation to the place of disposition. It may be paid for by the unit of cubic metre - kilometre (volume - distance) based on the excavation quantity (m³) and the distance (km) between the centres-of-gravity of the excavation and disposition areas, but more commonly, for smaller jobs, the costs for hauling the excavated material is usually lumped into the unit costs per metre for excavation.

(c) Compacted Embankments - includes supplying the labour and equipment for adding moisture or drying, compacting, and grading materials deposited in the embankment according to the drawings and specifications. It is normally paid for by the cubic metre as determined from the drawings or as-constructed
survey. The costs of obtaining the materials for compacting is not included in the item since their costs are covered under "Excavation".

(d) Select Filter Sand - includes supplying the materials, labour, and equipment for the supply and placement of select filter sand for the internal drainage system. It is normally paid for by the cubic metre as determined from the drawings.

(e) Riprap - includes supplying the materials, labour and equipment for the supply and placement of riprap slope protection and channel erosion protection. It is normally paid for by the cubic metre as determined from the drawings or post-constructed surveys.

(f) Bedding Gravel - includes supplying the materials, labour, and equipment for the supply and placement of bedding gravel on the upstream embankment slope and beneath riprap in discharge channels. It is normally paid for by the cubic metre as determined from the drawings or post construction surveys.

(g) Grassing - includes obtaining and placing topsoil from topsoil stockpiles, supplying grass seed, labour, and all equipment for grassing the downstream embankment slope and the auxiliary earth channel spillway. It is paid for by the square metre as determined from the drawings or post construction surveys.

Structure costs can be separated into the following items:

(a) Earthwork - includes excavation to required grade lines and backfill. For structures excavation is normally paid for under the excavation and compacted embankment items.

(b) Reinforced Concrete - includes supplying all the materials, labour and equipment to complete the concrete work including forming, reinforcing steel, concrete materials and mixtures, placing, consolidating, curing of concrete and removal of forms. It is normally paid for by the cubic metre for material volumes determined from the drawings.

(c) Pipes and Valves - includes supplying all the materials, labour and equipment for the supply and installation of pipes and valves through the embankment including CSP gatewells, slide gates and water works valves. It is normally paid for as a lump sum or based on price per lineal metre for lengths of pipeline determined from the drawings or measured in the field and includes valves, wet well, gates, etc. It is sometimes convenient to further breakdown the "Pipes and Valves" costs into supply of materials and installation.

(d) Riprap Plunge Pool Basin - includes supplying all the materials, labour and equipment for supply and placement of a riprap plunge pool basin at the pipe outlet. It normally
is paid for as a lump sum or based on the cubic metre for volumes determined from the drawings or post construction surveys. It is sometimes convenient to further break down the cost of "Riprap Plunge Pool Basin" into costs for riprap and costs for bedding gravel.

The determination of quantities for use in the cost estimates (quantity take-off) is performed using computation, graphical methods or computer methods. All work is to be documented on computation sheets, reviewed by another qualified experienced designer and filed in the Area Offices.

It is incumbent on the designers in each Area Office to establish contact with material suppliers and contractors and monitor project costs in their area in order to develop a schedule of prices for cost estimating specific to the particular area. The schedule should be reviewed and updated annually.
APPENDIX D
DESIGN EXAMPLE
APPENDIX D - DESIGN EXAMPLE

D-1 General

This section illustrates the application of this manual to a typical small dam project in the prairies. The documentation and explanation of this design example is referenced to the Project Flow Chart on Figure 1.4.2 and includes: notes on discussions and decisions with owner, design computations, and parameter selection; design input and output summaries; drawings and specifications. Some items and aspects of the design are discussed at length to clarify or emphasize certain aspects of the manual while other obvious or rudimentary items are only referenced for convenience and simplicity.

Although most dams that will be designed using this manual will be smaller than the one in this example, it was deemed appropriate to select a dam that would be large enough to demonstrate most of the concepts discussed herein. The methodology for the design of smaller projects would be the same.

In this example, a real land location, topography, and soils information have been used, but the project requirements and the names of the land owners have been made up.

D-2 Problem

An individual farmer, Joe Smith, desires to irrigate approximately 40 hectares (100 acres) in the NW 1/4 6-3-20W1. A natural drainage course is located along the west side of the quarter section. Mr. Smith has come to the local PFRA area office and requests assistance for his proposed project. The area manager assigned an experienced senior technician as the project designer to plan, investigate, and design the complete project using the PFRA "Small Dam Design and Construction Manual".

D-3 Given

The project designer meets with the project proponent to discuss the proposed project. At the end of the meeting the project designer prepares a note to the project file summarizing the meeting with the client and making an inventory of existing information. In this case the existing information is listed below.

Project: Joe Smith Dam

Date of first planning meeting with Owner: July 19, 1992

Location: NW 1/4 6-3-20W1

Use: Irrigation using sprinklers of 40 hectares (100 acres)

Previous Studies: the Provincial Department Agriculture has conducted soils, water quality tests and water demand studies. It was concluded that the soils
and water quality in the NW 1/4 6-3-20W1 are suitable for irrigation
and the annual water demand would be 220 dam³.

: similar types of irrigation projects have been
studied and constructed in the same area.

Mapping: 1:50 000 NTS

Soils Information: Water well logs, dugout excavation in NW 1/4-6-
3-20W1, borrow area excavation pit in SW 1/4-6- 3-20W1.

Owner Preference: desires dam location on NW 1/4 6-3-20W1

: is prepared to pay a moderate or reasonable
increase in capital cost in order to obtain a
relatively low-maintenance facility.

: may be willing to accept up to a 30% shortage
in satisfying the project demand

: desires an outlet structure of 10 l/s.

Scheduled date for preliminary site inspection and reconnaissance
study : August 2, 1992

Land Control : Project proponent owns all the land in
Section 6-3-20W1 on which the dam is to be
located and most of the land that would be
flooded by the reservoir.

Aerial Photos : A16277-95, A16277-94 (1 inch = 1320 feet)

D-4 Planning and Preliminary Design

Once the project objectives are defined, the available information
reviewed and the proponent request finalized, various types of
engineering investigations and studies are initiated. The engineering
activities are structured to provide an information base for design
decisions and details; develop design alternatives; communicate project
details and costs and confirm commitment with the project proponent;
and provide check points to ensure that the "Small Dam Design and
Construction Manual" is applicable for this particular project.

The steps in a hypothetical preliminary design are developed and
described in the following sub-sections.
D4A - Reconnaissance Inspection and Initial Assessment

1. Conduct site inspection and meeting on August 2, 1992 with project proponent.

2. Identify three potential dam site locations in NW 1/4 6-3-20W1. Site A (downstream site) has an estimated dam length of 110 m for a 8 m high dam. Site B (middle) has an estimated dam length of 130 m and Site C (upstream) has an estimated dam length of 120 m. See reference sketch Figure D.1 and stereogram in Figure D.3.

3. Project proponent confirms desire to irrigate 40 hectares (100 acres) requiring an annual summer season water demand of 220 dam$^3$ and may be willing to accept shortages in 30% of the years.

4. The following observations were made on the watershed and topographic land form characteristics during the inspection.

   - vegetation coverage: 75 cultivation 15 pasture 10 trees

   - well developed downstream channel

   - first downstream road crossing from proposed damsite

     : 1-900 mm diameter CSP culvert

     : road embankment approximately 1 m above stream flood plain

     : before water would overtop the road, a pool of water having a width of 50 m and a length of about 200 m would form against the road embankment in the valley upstream of the road

   - second downstream road crossing from proposed damsite

     : four 600 mm diameter CSP culverts and a 1300 2600 m concrete conduit

     : road embankment approximately 2 m above stream flood plain

     : at the top of road elevation ponding would occur over an area approximately 600 m wide by 200 m long before overtopping

   - railway crossing

     : railway embankment approximately 2 m above stream flood plain and would pond an area approximately 1000 m wide and 200 m long before overtopping
farmsteads and building are over 10 m above flood plain

5. On 1:50,000 NTS map delineate gross and effective drainage areas for the middle damsite location. Gross drainage area is 38 km² and effective drainage area is 34 km². Refer to Figure D.2.

Since all the identified dam sites are located in close proximity, it is reasonable to assume that their effective and gross drainage areas are approximately the same.

6. Identify dam site (NW 1/4 of 6-3-20W1) in region #1 on Figure 2.4.8 "Runoff - Draft Regions for the Prairie Provinces". In this region the median annual unit runoff is 20 dam³/km² determined from Figure 2.4.7 "Median Annual Unit Runoff from the Prairie Provinces".

The approximate median annual runoff at any damsite located in the NW 1/4 6-3-20W1 is 20 dam³/km² x 34 km² = 680 dam³ per year. The required annual demand (annual draft for May - August draft period) as a percent of the median annual runoff is 220 x 100 \[\frac{220}{680}\] = 32%.

Determine the reservoir capacity required in terms of percent of median annual runoff for firm annual draft and for shortages in 30% of the years during a May - August draft period using Figure 2.4.9 "Storage - Draft Curves for Region No. 1 to be 180% and 43% respectively. The required storage is approximately 1.80 x 680 or 1220 dam³ for a firm annual draft and .43 x 680 or 292 dam³ for shortages in 30% of the years for a May - August draft period. Therefore this site has sufficient water supply potential to satisfy a May - August demand of 220 dam³ annually with shortages in less than 30% of the years.

It is noted that for a storage at FSL of 292 dam³, a dam project with an embankment height less than 8 m and a storage at top of dam elevation of less than 400 dam³ may be possible and may be designed using "Small Dam Design and Construction Manual". However a dam with a storage at FSL of 1220 dam³ is outside the scope of the "Small Dam Design and Construction Manual". Before a decision could be made as to how this project should be developed the following questions would have to be answered:

1. Would it be feasible technically to develop 1220 dam³ of storage at these sites?
2. What would be the cost and would the Owner be willing or able to pay his share?
3. What other effects would the larger dam have? (on upstream and downstream land?)
4. Besides the no shortage option and 30% shortage option, what are the costs and effects of the alternatives in between?

For the purposes of this example, we will assume that a larger project than one having a storage of 292 dam³ is not feasible, or rejected for some other reason.

7. Prepare a preliminary cost estimate for site A (Downstream site) as it has a shorter length and may provide the most storage within NW 1/4 6-3-21W1. Additional sites may be assessed when topography maps are available. Based on local experience on a similar size project (an 8 m height dam with a spillway and riparian) costs are approximately $100 per m of dam length per height of dam. Using this costing basis, the Joe Smith project is estimated to cost 110 x 8 x 100 or $90,000. This estimate is an order of magnitude type of approximation based on previous similar types of projects not specific project details. The designer should prepare and update annually various cost estimating parameters so that these types of estimates can be readily prepared.

8. Contact the provincial water licensing agency to inquire if a water license could be obtained for this project. Normally this information is readily available from provincial local area representatives. In this case a water license would be granted.

9. Check ownership of land affected by project including downstream sections. Project proponent owns land at the proposed dam site locations and along the downstream channel for approximately 0.8 km. Flood easements for reservoir are available below the edge of present cultivation from T. Black owner of SW 1/4 6-3-20W1.

10. Prepare a preliminary design summary report and send to project proponent asking if he wants to proceed with project investigations.
August 14, 1992

Mr. Joe Smith
P. O. Box 321
Boissevain, Manitoba

Dear Sir:

Re: Joe Smith Dam Project

Please find enclosed a preliminary project summary report on the above noted project, prepared by P. Jones. Please advise if you are still interested in constructing this project so that we are able to schedule the additional necessary engineering studies. At the completion of these studies, another report will be submitted summarizing the project studies and recommending construction or cancellation of the project.

D. Brown
Area Manager, PFRA
Brandon, Manitoba
Preliminary Project Report

Date: August 10, 1992

By: Pete Jones

Project: Joe Smith Dam

Location: Three potential sites in NW 1/4 6-3-20W1

Use: Irrigation

Demand: 220 dam$^3$ with shortages in 30% of the years and a May - August demand period.

Preliminary Storage requirement: 290 dam$^3$

Land Control: J. Smith, project proponent owns the land at the proposed dam location and some 0.8 km downstream of the dam. It would be necessary to obtain a flood easement from T. Black who owns SE 1/4 1-3-21W1, a portion of which land would be flooded by the proposed reservoir.

Preliminary Estimated Cost: $90,000 (based on similar projects)

Other Studies: Studies by Provincial Agriculture Department have concluded that the soils and water are suitable for irrigation and that a water license for 220 dam$^3$ is available.

Recommendation: A preliminary assessment of this project indicates that it is feasible and that further engineering investigations are justified if the project proponent wishes to proceed.
To Whitewater Lake

Road to Whitewater

NE¼ 1-3-21 WI
Owner - T. Black

NW¼ 6-3-20 WI
Owner - J. Smith

Farm #1

Temporary Road

Site A

Earth Spillway

Irrigation Area

Site B

Site C

Potential Borrow Area

Existing Small Dam

X - Proposed Test Pit

SE¼ 1-3-21 WI
Owner - T. Black

SW¼ 6-3-20 WI
Owner - J. Smith

PROJECT AREA SKETCH

FIGURE: D.1
STEREORAM

FIGURE: D.3
1. With the aid of aerial photographs two farms and three crossings are identified and noted on 1:50 000 NTS topography maps. Refer to Figure: D.4.

2. A site inspection was conducted and the following observations noted:

(a) Farm #1 is over 10 m above the adjacent floodplain and not affected by a dam failure.

(b) The first downstream crossing is a low level crossing for a minor farm access road. There is one 900 mm diameter CSP culvert through road embankment. The road embankment is approximately 1 m above average floodplain and would retain a flooded triangular area approximately 50 m wide and 20 m long before overtopping.

(c) Farm #2 is over 10 m above adjacent floodplain and not affected by a dam failure.

(d) The second downstream road crossing is a main road crossing consisting of 4 - 600 mm diameter CSP culverts and a 1300 x 2600 concrete conduit. The road embankment is approximately 2 m above the average floodplain and would contain a flooded area approximately 600 m wide and 300 m long before overtopping.

(e) The third downstream crossing is the CPR railway crossing. The railway embankment is approximately 2 m above the average floodplain and would contain a flooded rectangular area approximately 1000 m wide x 200 m long before overtopping.

3. Computation of flood volumes retained by crossings.

(a) The volume contained upstream of the first crossing is approximately 0.5 (50 x 200 x 1) or 5000 m³.

(b) The volume contained upstream of the second crossing is approximately 2 x 600 x 300 or 360 000 m³. The total volume contained by road embankments upstream of the second road embankment is approximately 360 000 + 5000 or 365 000 m³.

(c) The volume contained upstream of the railway crossing is approximately 2 x 1000 x 200 or 400 000 m³.

4. Computation of flood damage costs

(a) There was no flooding of any downstream farms, therefore there are no damages to farmsteads from flooding due to a dam failure.
(b) For a reservoir of approximately 290 dam assume the top of dam elevation storage is 400 dam$^3$. This should be checked when the top of dam elevation is established. As the first downstream crossing only retains approximately 5 dam$^3$ compared to 400 dam$^3$, the road would be overtopped and require maintenance as a result of flooding from a dam failure. Repairs of this nature to a low class of road would be minimal.

The second road crossing may not be overtopped as the total volume of storage behind embankment crossings upstream of Crossing #2 is approximately 365 dam$^3$ which is close to the reservoir storage at the top of dam elevation. Therefore, there is no damage to the second crossing. Downstream from crossing #2 the flow would be primarily in the stream channel and no flooding or damage would occur to Crossing #3.

5. It is concluded that the hazard potential for this project is low (C) as there is no loss of life, damages due to flooding are less than $100,000 and the loss of the dam and its benefits are less than $200,000. It is noted that the land downstream of the dam to the second crossing is owned by the project proponent. Refer to Figure: D.5 for the "Hazard Potential Summary".
HAZARD POTENTIAL IMPACTS

FIGURE D.4
# HAZARD POTENTIAL ASSESSMENT SUMMARY

<table>
<thead>
<tr>
<th>Project Title / Owner:</th>
<th>Joe Smith Dam</th>
<th>Location:</th>
<th>NW 4 6-3-20W1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Designer:</td>
<td>P. Jones</td>
<td>Date:</td>
<td>92/08/09</td>
</tr>
</tbody>
</table>

![Downstream Schematic Map]

- **Estimated Embankment Height:** 8 m
- **Estimated Storage at Top of Dam:** 400 dam³
- **Office Studies:** ☑ yes  ☐ no
- **Loss of Life:** ☐ yes  ☑ no
- **Reference Map:** ☑ yes  ☐ no
- **Date of Field Inspection:** 92/08/08
- **Estimated Flood Damage (1980 $):** UNDER $109,000
- **Estimated Other Damage (1980 $):** UNDER $209,000
- **Discussion with Project Proponent:** ☑ yes  ☐ no
- **Hazard Potential Rating:** C ☑ greater than C ☐

---

**Figure D.5**
D4C  Geological Damsite Description

A geological damsite description summary sheet was prepared using the following information:

(a) Twelve backhoe test pits constructed at the project site August 17, 1992.


(h) Dugout excavation in NW 1/4-6-3-20W1.

(i) Old borrow pit excavation in SW 1/4 6-3-20W1.

(j) Water well logs.

Refer to Figure D.6 for the geological damsite description.
**GEOLOGICAL DAMSITE DESCRIPTION - SUMMARY SHEET**

Project Title / Owner: **JOE SMITH DAM**  
Location:  

Designer: **P. JONES**  
Date: **92/08/19**  

Site or Centreline Designations: **DAM E**

Site comments based on interpretation of the following information:

- **Field Exploration Data**:  
  - Soil Maps and Reports: ✅  
  - Subsurface Investigation: ✅

- **Local Water Well Logs**: ✅  
  - Geological Maps and Reports: ✅  
  - Air Photo Interpretation: ✅

A. **Abutments**

- **Geological Unit(s) forming abutments**: Pre-valley: ✅  
  - Post valley: ✅
  - Clay: ❌  
  - Silt: ❌  
  - Sand: ❌  
  - Gravel: ❌  
  - Till: ✅  
  - Shale: ❌  
  - Sandstone: ❌

  Other units present:

- **Evidence of abutment instability**: None ✅  
  - Dormant: ❌  
  - Active: ✅

- **Evidence of natural seepage**: None ✅  
  - Present: ❌

- **Estimated valley slopes**: 4:1

B. **Foundation**

- **Geological Unit(s) immediately underlying valley floor**: Pre-valley: ✅  
  - Post valley: ✅
  - Clay: ❌  
  - Gravel: ❌  
  - Shale: ❌  
  - Silt: ✅  
  - Till: ❌  
  - Sand: ✅  
  - Sandstone: ❌

  Other units present:

- **Estimated maximum thickness**: 1.5 m

- **Evidence of high water table**: None ✅  
  - Present: ❌

- **Evidence of salt concentration on surface**: None ✅  
  - Present: ❌

C. **Reservoir**

- **Cultural features affected by reservoir**:  
  - Buildings: None ✅  
  - Present: ❌  
  - Roads: None ✅  
  - Present: ❌

- **Evidence of valley side instability**: None ✅  
  - Dormant: ❌  
  - Active: ✅

D. **Materials of Construction**

<table>
<thead>
<tr>
<th>Material</th>
<th>Local (within 200 m)</th>
<th>Within 1 km</th>
<th>Unknown or &gt;1 km</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay or Till</td>
<td>✅</td>
<td>✅</td>
<td>✅ 8 km</td>
</tr>
<tr>
<td>Sand and Gravel</td>
<td>✅</td>
<td>✅</td>
<td>✅ 5 km</td>
</tr>
<tr>
<td>Cobbles and Boulders</td>
<td>❌</td>
<td>❌</td>
<td>❌</td>
</tr>
</tbody>
</table>

E. **Schematic Valley Cross-Section at Embankment Centreline** (attach additional sheets for reservoir cross-sections)
D4D  Geotechnical Assessment

The geotechnical assessment determines the geotechnical suitability of a particular site for construction of a dam. The assessment is made based on previous information (reconnaissance studies, geological studies, other available information) and a limited subsurface investigation conducted by the project designer.

Generally in planning for a subsurface investigation, information from the reconnaissance studies is used and only the most favorable site is studied. However, one or more additional sites may be studied for general interest, confirmation of soil units, or if the first site is obviously unsuitable. For the J. Smith project, 12 test pits (backhoe) will be excavated (Refer to Figure D.1, Project Area Sketch). Test pits will be excavated at the following locations for Site A:

- 1 on dam centerline on the valley floor, south of the existing stream which could be the location of a drop inlet spillway;
- 1 on each side of the valley at the junction of the valley floor and valley slope on dam centerline;
- 1 on each abutment approximately 6 - 7 m above valley floor along dam centerline.
- 1 on the valley floor along centerline of dam site C;
- 1 in the middle and at each end of the earth spillway; and
- 3 located in a potential borrow area in the NW 1/4 6-3-20W1 and SW 1/4 6-3-20W1.

The final location of the test pits will be determined in the field at the time of the investigation and surveyed during topographical surveys. The test pits are to be excavated to a minimum of 4 m. Samples are obtained and a log of each test pit is prepared. After laboratory analysis of the soil samples, the soil logs are revised and updated if necessary. Copies of the soil log are included in the geotechnical assessment. Refer to Figures D.7A and D.7B.

Based on the Table 2.5.2 "Classification of Foundation and Embankment Materials for Homogenous Small Storage Dams" in the Small Dam Design and Construction Manual and the information obtained from the soils investigation, the geotechnical suitability of the foundation and embankment materials are determined. It is noted that projects which do not have good foundation or embankment materials are outside the scope of the "Small Dam Design and Construction Manual". Refer to Figures D.8A and D.8B for the Geotechnical Assessment Report".

Concrete (ready mix) and pervious materials are available from suppliers in Boissevain. Rock riprap is available on the project proponent’s farmstead.
# PFRA Testhole Log

**Project:** JOE SMITH DAM  
**Site:** NW 1/4 6-3-20W1  
**Hole No.:** 1-5  
**Elev.:** m  
**Hole Location:**  
**Classified by:** P. JONES  
**Date:** AUG 21/92  
**Static Water Level:** m  
**Time:** Date  
**Test Installation:**  
**Page 1 of 2**  
**Comments:**  

### Depths - m  
**(Underline sample depth)**

<table>
<thead>
<tr>
<th>Depth Range</th>
<th>Soil Classification</th>
<th>Description, Characteristics, Abnormal Conditions, Sample Type, Water Data</th>
<th>Sample Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.2 - 4</td>
<td>C1</td>
<td>Till, 0.1 gravelly lag @ surface</td>
<td>Sealer #1</td>
</tr>
<tr>
<td>0 - 0.2</td>
<td>TOPSOIL</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.2 - 4</td>
<td>Cl</td>
<td>Till</td>
<td>Sealer #2</td>
</tr>
<tr>
<td>0 - 0.6</td>
<td>Cl Cl1</td>
<td>Floodplain</td>
<td>Sealer #3</td>
</tr>
<tr>
<td>0.6 - 1.5</td>
<td>SM</td>
<td>Alluvial (water @ m)</td>
<td>Sealer #4</td>
</tr>
<tr>
<td>1.5 - 4</td>
<td>C1</td>
<td>Till</td>
<td></td>
</tr>
<tr>
<td>0 - 0.2</td>
<td>TOPSOIL</td>
<td></td>
<td>Sealer #5</td>
</tr>
<tr>
<td>0.2 - 4</td>
<td>C1</td>
<td>Till (minor sand lenses)</td>
<td></td>
</tr>
<tr>
<td>0 - 0.2</td>
<td>TOPSOIL</td>
<td></td>
<td>Sealer #6</td>
</tr>
<tr>
<td>0.2 - 4</td>
<td>C1</td>
<td>Till</td>
<td></td>
</tr>
</tbody>
</table>

**NOTE:** All samples should be properly labelled with project, site, hole no., sample depth, sample classification and date.

**FIGURE D.7A**
## PFRA TESTHOLE LOG

### LOCATION SKETCH

(SEE PAGE 1)

**Project**

**Site**

**Hole No.** 6-12  **Elev.**

**Hole Location**

**Classified by** Date

**Static Water Level**  **Time**

**Test Installation** Date

**Page** 2 of 2  **Comments:**

### Soil Log

<table>
<thead>
<tr>
<th>Depths - m</th>
<th>Soil Classification</th>
<th>Soil Description, Characteristics, Abnormal Conditions, Sample Type, Water Data</th>
<th>Sample Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>TEST PIT 6</td>
<td>0 - 0.2  C1</td>
<td>TOPSOIL  TILL - GRAVELLY LAG  @ 1</td>
<td>Sealer #7</td>
</tr>
<tr>
<td>0.2 - 4</td>
<td>0.1  C1</td>
<td>TILL</td>
<td></td>
</tr>
<tr>
<td>TEST PIT 8</td>
<td>0 - 0.2  C1</td>
<td>TOPSOIL  TILL - GRAVELLY LAG  @ 1</td>
<td>Sealer #7</td>
</tr>
<tr>
<td>0.2 - 4</td>
<td>0.1  C1</td>
<td>TILL</td>
<td></td>
</tr>
<tr>
<td>TEST PIT 9</td>
<td>0 - 0.2  C1</td>
<td>TOPSOIL  TILL - GRAVELLY LAG  @ 1</td>
<td>Sealer #7</td>
</tr>
<tr>
<td>0.2 - 4</td>
<td>0.1  C1</td>
<td>TILL</td>
<td></td>
</tr>
<tr>
<td>TEST PIT 10</td>
<td>0 - 0.2  C1</td>
<td>TOPSOIL  TILL - GRAVELLY LAG  @ 1</td>
<td>Sealer #7</td>
</tr>
<tr>
<td>0.2 - 4</td>
<td>0.1  C1</td>
<td>TILL</td>
<td></td>
</tr>
<tr>
<td>TEST PIT 11</td>
<td>0 - 0.2  C1</td>
<td>TOPSOIL  TILL - GRAVELLY LAG  @ 1</td>
<td>Sealer #7</td>
</tr>
<tr>
<td>0.2 - 4</td>
<td>0.1  C1</td>
<td>TILL</td>
<td></td>
</tr>
</tbody>
</table>

**NOTE** - All samples should be properly labelled with project, site, hole no., sample depth, sample classification and date.

**FIGURE D.7B**
### Foundation Conditions

<table>
<thead>
<tr>
<th>Location</th>
<th>Soil Types</th>
<th>Depth of Groundwater (m)</th>
<th>Potential Problems</th>
<th>Suitability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Valley Bottom</td>
<td>C1 (TILL)</td>
<td>NEAR CHANNEL</td>
<td>NONE</td>
<td>GOOD</td>
</tr>
<tr>
<td>Left Abutment</td>
<td>C1 (TILL)</td>
<td>DRY</td>
<td>NONE</td>
<td>GOOD</td>
</tr>
<tr>
<td>Right Abutment</td>
<td>C1 (TILL)</td>
<td>DRY</td>
<td>NONE</td>
<td>GOOD</td>
</tr>
<tr>
<td>Upstream</td>
<td>C1 (TILL)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Downstream</td>
<td>C1 (TILL)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Final Assessment and Comments:**

APPROXIMATELY 150 mm TOPSOIL OVER AREA AND 1500 mm ALLUVIAL MATERIAL IN VALLEY BOTTOM, OVERALL SUITABILITY IS **GOOD**

### Borrow Materials

<table>
<thead>
<tr>
<th>Location</th>
<th>Soil Types</th>
<th>Water Content</th>
<th>Optimum Water Content</th>
<th>Suitability</th>
</tr>
</thead>
<tbody>
<tr>
<td>EARTH SAVY TP 6,7,8</td>
<td>C1 (TILL)</td>
<td>(TP7) 14%</td>
<td>16%</td>
<td>GOOD (ZONE 2)</td>
</tr>
<tr>
<td>TP 10,11,12</td>
<td>C1 (TILL)</td>
<td>(TP11) 17%</td>
<td>16%</td>
<td>GOOD (ZONE 1)</td>
</tr>
</tbody>
</table>

**Final Assessment and Comments:**

BORROW MATERIALS ARE **GOOD**
NOTE: Depth of Test Pit = 5m
D4E  Topographical Survey

Plan and conduct surveys for preparation of topographic maps. Survey all areas bounded by an elevation at least 10 m above the channel bottom of the dam site under consideration (Sites A and C). Tie in soil test pit location and the location of proposed Sites A, B and C.

Prepare storage capacity and flooded area versus elevation curves for Site A and C. Refer to Figure D.9. The reference topographic plan and computations for storage capacity and flooded area curves are standard items and their preparation is not included in this example.

D4F  Hydrology

Review and confirm the water supply potential and the required storage for Site A. (As noted previously the water supply potential and storage requirements at Site C is approximately equal to that at Site A.

The annual median runoff is 680 dam$^3$/year for Site A located in NW 1/4 6-3-20W1 with an effective drainage area of 34 km$^2$. For an annual demand of 220 dam$^3$ with shortages in 30% of the years over a May - August demand period the required storage is 292 dam$^3$ using the curves in Figure 2.4.9 "Storage - Draft Curves for Region No. 1.

Referring to the storage capacity curves for Site A (Figure: D.9) a Full Supply Level (FSL) of 528.7 m is required to provide a storage of 292 dam$^3$. Referring to the storage capacity curves for Site C, the required FSL is 530.2 m.

The flood potential for Site A or site C is now determined.

1. The effective drainage area as determined previously is 34 km$^2$.

2. The drainability of the basin is estimated based on Table: 2.4.1 "Drainability Factors Selections Chart", 1:50 000 topographic (NTS) maps and observations from site inspections.

   (A) Basin Slope (DF1): the weighted average slope throughout the basin is approximately 0.01. Therefore DF1 is 0.4.

   The determination of the weighted average basin slope using 1:50 000 NTS maps is shown in Figure: D.10 "Weighted Average Basin Slope".

   (B) Drainage Channel Development (DF2): basin has extensive channel development with essentially no significant depressed storage for approximately 90% of basin area with small lakes and slough in upper 10% of the basin area. Therefore estimate DF2 to be 0.2.

   (C) Basin Shape (DF3): basin is moderately long and thin. Therefore estimate DF3 to be 0.1.
(D) Vegetation Cover (DF4): the basin is cropped land and good pasture. Therefore DF4 is 0.05.

(E) Soil Texture (DF5): basin is primarily composed of loam soil. Therefore DF5 is 0.05.

The overall basin drainability is the summation of DF1 through DF5 or 0.80.

3. The index flood (1:2 flood) from Figure 2.4.1 "Index Flood Selection Graph" for an effective drainage area of 34 dam$^3$ and a basin drainability factor of 0.80 is 3.6 m$^3$/s.

4. The index flood multipliers from Figure 2.4.2 "Index Flood Multiplier Selection Graph" and a 34 km$^3$ effective drainage area are: 1:10 flood - 4.2; 1:20 flood - 6.6; 1:50 flood -10.2; 1:100 flood - 14.2.

5. The unadjusted flood peaks are determined by multiplying the index flood by the index flood multiplier and are listed below:
   - 1:10 - 15.1 m$^3$/s
   - 1:20 - 23.8 m$^3$/s
   - 1:50 - 36.7 m$^3$/s
   - 1:100 - 51.1 m$^3$/s

6. The flood peak adjustment factor due to rainfall amount for Site A is 1.2 using Figure 2.4.3. "Rainfall Isopleths 1:100 6 Hour Rain" and Figure 2.4.4. "Flood Peak Adjustment Factor Rainfall Component". (the 1:100 - 6-hour rain is 85 mm).

7. The flood peak adjustment factor due to antecedent soil moisture is 1.14 using Figure 2.4.5 "Net Evaporation Isopleths Average May - June Values" and Figure 2.4.6. "Flood Peak Adjustment Factor Soil Moisture Component. (The net evaporation for Site A is 30 mm).

8. The adjusted flood peaks for Site A are determined by multiplying the unadjusted peaks by the flood peak adjustment factors due to rainfall amount and antecedent soil moisture and are listed below:
   - 1:2 - 3.6 m$^3$/s
   - 1:10 - 20.7 m$^3$/s
   - 1:20 - 32.6 m$^3$/s
   - 1:50 - 50.2 m$^3$/s
   - 1:100 - 69.9 m$^3$/s

Refer to Figure D.11 for the project "Hydrology Summary Report".
NOTE: Channel Profile Plotted From
NTS (1:50,000) MAP (62, F1)

\[
\begin{align*}
S_1 &= \frac{32000 \times (0.0047)}{59000} + \frac{27000 \times (0.016)}{59000} \\
&= 0.00987 \approx 0.01
\end{align*}
\]
**Project Title / Owner:** JOE SMITH DAM  
**Location:** NW 1/4 6-3-20W1
**Designer:** P. JONES  
**Date:** 92/09/05

### General Site Characteristics
- **Effective Drainage Area:** 34 km²  
- **Gross Drainage Area:** 38 km²

### Watershed Description
- **Watershed is approx. 14 km long and 3.5 km wide.** It is relatively steep with a well-defined channel system. The upper portion (10%) is flatter with some erosional storage. The area is under cultivation or used for pasture except for a small portion in upper basin which is treed.

### Schematic Watershed Map

### Water Supply Potential
- **Runoff-Draft Region:**  
- **Median Annual Unit Runoff:** 20 dam³/km²  
- **Median Annual Runoff:** 680 dam³  
- **Project Demand:** 220 dam³

#### a) Live Stream Diversion (No Reservoir)
- **Divertable Annual Volume:** 50% of the time NA dam³; 70% of the time NA dam³

#### b) Storage Condition
- **Reservoir capacity as a % of Median Annual Runoff:** 43%; **Reservoir Capacity:** 290 dam³

- **Firm Annual Draft (0% shortages):** (Jan. - Dec. Demand) NA dam³; (May - Aug. Demand) 60 dam³

- **Annual Draft with shortages in 30% of the years:** (Jan. - Dec. Demand) NA dam³; (May - Aug. Demand) 220 dam³

### Flood Potential
- **Overall Drainability Factor:** 0.6  
- **Index Flood Multiplier:**
  - (1:10) 4.2; (1:50) 10.2; (1:100) 14.2
- **Rainfall Adjustment Factor:**
  - (1:10) 1.2; (1:50) 1.2; (1:100) 1.2
- **Soil Moisture Adjustment Factor:**
  - (1:10) 1.14; (1:50) 1.14; (1:100) 1.14
- **Instantaneous Flood Peaks (m³/s):**
  - (1:10) 21; (1:50) 50; (1:100) 70

---

**Figure D.11**
D4G Embankment Design

The locations Site A and Site C are reviewed based on the investigations to date. The hazard potential is "low". The foundation and embankment materials are "good". It is noted with respect to the topographic map that the required FSL of Site C is just above the ridge area (elevation 530.0) on the east side of the site. For this condition the ridge area would not accommodate a conventional earth auxiliary spillway and would also require embankment material above the ridge area to meet freeboard requirements. The FSL for Site A is considerably below (1.3 m) the ridge area on the east side of the site and would conveniently accommodate an auxiliary earth spillway. It is concluded that Site C should be abandoned and only Site A be considered in further design studies.

An embankment section, shown in the project layout drawing, was selected based on the "Proposed Embankment Design Cross-Section", Figure 3.3.2. The embankment section includes: 3 m top width; 2.5:1 downstream side slope, 3:1 upstream slope with riprap slope protection; a horizontal and inclined filter system with a finger drain; foundation stripping and a key trench 1-1.6 m deep; and a homogeneous earthfill.

Based on the moisture conditions from the site investigation of 1% above optimum water content for the proposed borrow area and 2% below optimum water content for the earthcut spillway area, materials from these areas are to be used as impervious Zone 1 and random Zone 2, respectively, subject to assessment at the time of construction.

As the embankment is comprised of glacial till materials an unprotected flat embankment upstream slope was considered as an alternative to riprap upstream slope protection. Based on the criteria in Table 3.3.1 "Design Standards for Upstream Slope Protection" and for an assumed 8 m high dam, with a reservoir length less than 1 km and exposure condition "A", the riprap slope protection consists of a 200 mm thick rock layer (D50 of 130) and 150 mm thick bedding gravel layer placed on a 3:1 slope from top to bottom of upstream slope. The unprotected flat embankment slope selected is 5:1. The approximate cost of riprap slope protection is $160/m of width based on a $20/m³ for rock in place and $15.00/m³ for bedding gravel in place. The cost of additional embankment to provide a 5:1 upstream slope instead of a 3:1 upstream slope is approximately $192/m of width based on a $3.00/m³ for embankment fill in place. Therefore, for preliminary design use riprap upstream slope protection.

D4H Spillway System Design

Based on a review of the site location and discussion with the project proponent, a spillway system comprised of a CSP drop inlet spillway for the operating spillway and an earth auxiliary spillway is considered. A CSP drop inlet spillway will provide reliable operation and low maintenance for average year flows while the auxiliary earth spillway will provide capacity for larger less frequent flow with
possibly some maintenance requirement. In selecting a CSP drop inlet spillway, it was noted that this spillway type is characterized by a relatively low surcharge or head requirement at the design flow and this would facilitate locating an auxiliary earth spillway in the east ridge area. The south side of the stream canal would accommodate a CSP drop inlet spillway and provide a construction area under dry conditions. The spillway exit channel would be connected to the existing stream channel when the project is completed. The ridge area on the east side of the dam is a suitable site for an earth auxiliary channel.

Based on criteria described in Section 3.4.2 "Criteria", the operating spillway will be sized for design discharge corresponding to the 1:2 flood event with no provision for reservoir routing. The hydrology investigation determined that the 1:2 flood event provides a peak inflow of 3.6 m$^3$/s. From the Standard Drawing SD04 "CSP Drop Inlet Spillway", a structure with a conduit diameter of 900 mm would provide a discharge of 3.4 m$^3$/s at a head of 0.4 m which is approximately the flood out point. The safety of dam design flood (SDDF) is the 1:100 flood event. From the hydrological investigations, the peak inflow corresponding to this event is 70 m$^3$/s. Using the standard plan SDO1A Auxiliary Earth Channel Spillway", a channel section with a width, 120 m; length, 20 m; and reservoir head 0.6 m, would provide a discharge of approximately 70 m$^3$/s. By setting the crest of the drop inlet spillway at the FSL and the inlet elevation of the auxiliary earth spillway 0.4 m above FSL (at OSDF surcharge elevation) the flood surcharge for the SDDR is 1.0 m.

At this point in the investigation, the designer recognizes the fact that the auxiliary earth channel spillway has a length of only 20 m, which if the materials in the bed of the channel are erodible, does not provide much protection for the reservoir. What are the options?

1. Check bed material for erodibility, if highly erodible, could relocate spillway channel to obtain a longer length.
2. Could drop reservoir FSL, downsize project to get longer length at same location.
3. Could reduce the frequency of use by making operating spillway larger so that earth spillway does not operate as often; or
4. Accept the proposed design and provide maintenance as required.

Using the Figure 3.4.1 "Freeboard Allowance", the freeboard requirement above FSL for a site with a reservoir length of 0.5 km or less, and a dam height of 7 - 8 m is approximately 1.0. For this freeboard condition and a FSL elevation of 528.7 m at top of dam elevation of 529.7 would be required. The flood freeboard above the maximum surcharge elevation (SDDF) is 0.3 from Figure 3.4.1 "Freeboard Allowance". For this freeboard condition (1.0 m flood surcharge and 0.3 m flood freeboard) the top of dam elevation would be 530.0 m. The freeboard condition with highest top of dam elevation governs,
therefore, the top of dam is 530.0 and the maximum height of dam is 8 m. The crest elevation of the drop inlet spillway is 528.7. The inlet elevation of the auxiliary earth spillway is 529.1. It is noted at this point in the work that at the design top of dam elevation (530.0) the height of dam is equal to or less than 8 m; however, the maximum storage is 460 dam^3 which is greater than 400 dam^3 specified for application of the small dam design manual. After consultation with the Area Manager, it was decided to proceed with the design using the Small Dam Design and Construction Manual but have work reviewed by a qualified professional engineer. It was observed that for this project only the storage guideline was not satisfied.

A CSP Drop Inlet Spillway location was selected at Station 95.00 based on the following considerations:

- construction to be performed in the dry and not affected by existing stream
- separate location from riparian structure to minimize the effects of the failure of one structure to the other.
- Provide a practical, economical connection of the spillway exit channel with the existing stream channel
- minimize excavation associated with spillway
- provide good foundation
- provide suitable topography and layout alignment.

The average elevation of the existing stream bank was estimated to be approximately 523.1 (1.2 m above stream bottom) at the dam location. Select an exit channel invert elevation one spillway conduit diameter (0.9 m) below the stream bank elevation or 522.2. Connect the spillway and the existing stream channel with an exit channel consisting of a bed width equal to the plunge pool width TP5 or 1.2 m, bed slope of 0%, and side slope of 3:1.

**D4I Riparian Outlet**

Based on a review of the site location and the project proponent requirements, a 100 diameter PVC Riparian Structure - Type I from the Standard Drawing SD10 was selected and located at station 75.00 m. This structure would provide the required demand of 10 l/s with a head of approximately 0.5 m. The structure was located on the north side of the existing stream to provide separation from the CSP drop inlet spillway and to provide dry construction conditions. The riparian structure is connected to the stream channel by a channel with a bed elevation of 522.2, bed width of 0.5, bed slope of 0% and side slopes of 3:1. The bed elevation of the exit channels for the riparian structure and the CSP Drop Inlet Spillway are the same. The slope of the pipe is set to provide a drop of 300 mm from inlet to outlet for drainage.
The project designer checks the preliminary design. Preliminary drawings are prepared in order to describe the project and permit the determination of approximate construction material quantities. Preliminary drawings would include (a) a location and site layout drawing comprising a site location map; a topographic plan and structure locations; dam profile and cross-section showing soil logs; structure profiles; (b) CSP Drop Inlet Spillway drawing; and (c) PVC Riparian Structure - Type I drawing. Using the approximate material quantities and unit prices based on local experience, a preliminary cost estimate is prepared. For this example the following cost estimate summary has been prepared as shown in Table D.1.

The preliminary design (computation and drawings) were reviewed by a qualified professional engineer and found to be satisfactory and appropriate.

A "Preliminary Design Report" is prepared and sent to the project proponent. A covering letter would inquire if the project proponent wishes to proceed with the project.
<table>
<thead>
<tr>
<th>Item</th>
<th>Quantity</th>
<th>Unit Price</th>
<th>Extension</th>
</tr>
</thead>
<tbody>
<tr>
<td>Excavation</td>
<td>17000 m³</td>
<td>100/m³</td>
<td>17 000</td>
</tr>
<tr>
<td>Compacted Embankment (Zone 1 &amp; 2)</td>
<td>12000 m³</td>
<td>0.75/m³</td>
<td>9 000</td>
</tr>
<tr>
<td>Select Filter Gravel (Zone 3A)</td>
<td>740 m³</td>
<td>12.00/m³</td>
<td>8 900</td>
</tr>
<tr>
<td>Bedding Gravel (Zone 3B)</td>
<td>620 m³</td>
<td>8.00/m³</td>
<td>5 000</td>
</tr>
<tr>
<td>Riprap (Zone 4 and 5)</td>
<td>1240 m³</td>
<td>15.00/m³</td>
<td>18 600</td>
</tr>
<tr>
<td>Concrete</td>
<td>12 m³</td>
<td>200/m³</td>
<td>2 400</td>
</tr>
<tr>
<td>Pile Drilling</td>
<td>1</td>
<td>Lump sum</td>
<td>200</td>
</tr>
<tr>
<td>100 PVC Pipe</td>
<td>46.70 m</td>
<td>12.00/m</td>
<td>600</td>
</tr>
<tr>
<td>900 CSP Pipe</td>
<td>31.73 m</td>
<td>10.500/m</td>
<td>3 400</td>
</tr>
<tr>
<td>1800 CSP Riser Section (including 2000 and 2400 exterior sections)</td>
<td>1 Lump sum</td>
<td>4 300</td>
<td></td>
</tr>
<tr>
<td>150 CSP End Sections (4 m long)</td>
<td>1</td>
<td>Lump Sum</td>
<td>200</td>
</tr>
<tr>
<td>100 Gate Valves</td>
<td>1</td>
<td>Lump Sum</td>
<td>400</td>
</tr>
</tbody>
</table>

Subtotal                                  | 70 000    |

Contingencies 15%                          | 10 500    |

Total                                     | 83 000    |
Project: Joe Smith Dam

Location: NW 1/4 6-3-20W1

Use: Irrigation

Demand: 185 dam$^3$ with shortages in 30% of the years and a May-August demand period.

Storage Requirement: 290 dam$^3$

Land Control: J. Smith, project proponent owns the land at the proposed dam location and 0.8 km downstream from the dam. Flood easements are required and available from T. Black, owner of land in SE 1-3-21W1, a portion of which would be inundated by the proposed reservoir.

Project Details: The enclosed preliminary project drawings (Project Location, Layout and Details; CSP Drop Inlet Spillway; and PVC Riparian Structure Type I) describe the project and its components. (The drawings which would be appended to a report such as this are not included here with this example since they are similar to those which have been prepared and are included as construction drawings with the specifications for this example).

The foundation and embankment materials have been classified as "good" based on geotechnical investigation.

This project has a low hazard potential rating.

A homogenous earth fill with rock riprap upstream slope protection, internal drainage system and maximum height of 8 m impounds a reservoir with a storage volume of 290 dam$^3$ at elevation 528.7 m (FSL) and 460 dam$^3$ at the top of dam elevation 530.0 m.

The spillway system consists of a CSP Drop Inlet Spillway which passes the 1:2 inflow flood event of approximately 3.6 m$^3$/s at a surcharge elevation of 529.1 m and a 120 m wide auxiliary earth spillway which passes the 1:100 inflow flood event of approximately 70 m$^3$/s at a surcharge elevation of 529.7 m.

The 100 mm diameter PVC riparian outlet structure provides an approximate discharge of 10 l/s at a head of 0.5 m.
Based on preliminary estimates of material quantities, unit prices and a contingency of 15% excluding engineering costs, the project is estimated to cost approximately $83,000.

Recommendation: The Joe Smith Dam is a technically feasible project and could be constructed at an estimated cost of $83,000.
D5 Final Design and Construction

All aspects of the preliminary design are reviewed and confirmed.

- water rights license could be obtained
- hazard potential is "low"
- foundation and embankment materials are "good"
- dam height is 8 m and storage at top of dam is 460 dam$^3$

The proposed projects includes the embankment, 900 mm diameter CSP Drop Inlet Spillway, 120 m wide auxiliary earth spillway and a 100 mm diameter PVC Riparian type 1 structure.

It is noted that a qualified professional engineer has reviewed the design and approved application of the "Small Dam Design and Construction Manual" although the storage at top of dam (460 dam$^3$) is greater than specified in the manual (400 dam$^3$).

The following optimization alternatives were considered, however, they were found to be less cost-effective or desirable than the above proposed project and are not described in detail.

- 900 mm diameter CSP sloping pipe spillway and 200 m wide auxiliary earth spillway
- 900 mm diameter CSP Drop Inlet spillway and a 200 m wide auxiliary earth spillway

It is noted that although the 900 mm diameter CSP Drop Inlet spillway and 200 m wide auxiliary earth spillway alternative provides a slightly lower top of dam elevation and cost, it is considered desirable to have the top of dam elevation at or above the elevation of the ridge area on the east side of the dam. This will provide an additional margin of safety to the embankment as flow not contained in the excavated earth will flow over the existing natural ridge area before overtopping the dam.

In this case the preliminary design proposed was also the final design proposed. The pertinent drawings and specifications are selected and prepared. A cost estimate of the final design proposal is prepared. The final design is approved by another technologist or the Area Manager.

The final design proposal is discussed and accepted by the project proponent.
A project review package is sent to the designated PFRA authority for approval. The review package consists of:

- copy of request from project proponent and summary of project demands
- copy of design inputs including - hazard potential assessment, geological damsite description, geotechnical assessment, and hydrology summary report
- copy of drawings and specifications
- copy of cost estimate.

Upon completion of approval by the PFRA authority, the necessary funding and provincial approvals and applications are completed. After all the funding arrangements are completed and construction approvals are received, a set of drawings and specification are prepared for project construction as shown at the end of this section.

During construction the project designer should provide site supervision paying particular attention to:

- excavation of key trench and materials encountered
- stripping and foundation preparation to confirm design assumptions from test pits that foundation is good or fair
- excavations for structures to confirm foundation condition
- construction of embankment to insure knitting of layers, adequate densities and moisture contents, proper material for construction
- construction of filter layer to insure pervious uncontaminated layers
- water-tight pipe joints adequately restrained and able to resist pull-out forces
- proper compaction of backfill against pipes and conduits to prevent piping and material transport
- correct elevations and location of earth cut spillway, structures and dam embankment

After construction is complete, as constructed drawings are prepared. Water right license procedures are completed. An operation and maintenance manual is prepared and presented to the project owner.
JOE SMITH DAM

SPECIFICATION/DRAWINGS
SPECIFICATION FOR EARTHWORK

Excavation

All excavation for the work shall be performed by the Contractor and shall include but not necessarily be limited to the following excavation operations:

1. Stripping shall include removal of vegetation, topsoil and all material unsuitable for forming foundations for the embankment or structures. Stripping shall also include removal of vegetation and topsoil from sources of borrow material and the earthcut spillway. Stripping material suitable for use as topsoil in grassing operations shall be stockpiled for later use at convenient locations so as not to interfere with the work. Stripping material not suitable for any purpose shall be wasted at locations which will not interfere with construction or operation of the dam.

2. Excavation for the key trench involves removal of soft, pervious, crumbly or fractured foundation soil to expose an impervious contact face. Structure excavation involves removal of material to the desired elevation upon which to place the structure. Structure excavations shall not be made deeper than shown on the drawings unless specifically instructed to do so by the Owner. Materials suitable for use in the embankment shall be placed directly into the embankment or stockpiled for later use. Unsuitable materials shall be wasted.

If unsuitable material is exposed upon completion of the excavation for the foundation of a structure, the Contractor shall notify the Owner or inspector and shall not proceed with the construction of the structure until directed by the Owner or inspector.

3. Excavation for the earthcut spillway involves the removal of material to produce the desired cross-sectional shape at the specified elevation as shown on the drawings. Material suitable for use in the embankment shall be placed directly in the embankment or backfill areas or stockpiled for later use. Unsuitable materials shall be wasted.

4. Excavation in the borrow areas involves obtaining suitable material for use in the embankment or backfill from designated borrow areas and placing the suitable materials directly in the embankment or backfill. In cases where unsuitable material must be removed to expose suitable material, the unsuitable material shall be wasted.
Excavation includes loosening, digging, loading, hauling and disposal of excavated materials at the designated location or point of use. Excavations shall be planned to provide the most suitable materials for construction and to minimize the handling of material. Permanent excavation slopes shall be constructed at 3:1 and maintained in a neat, uniform condition. Temporary excavation slopes shall not be steeper than permitted by the Worker's Compensation Board.

Foundation Preparation

The entire foundation surface shall be stripped of topsoil and organic matter. Crumbly or fractured foundation soil in the key trench shall be removed to the depth shown on the drawings, determined in the field, or specified in these specifications. If pervious material zones or soft material occurs below the stripped depth or crumbly, fractured soil occurs in the key trench below a depth of 2 m the owner or inspector should be notified and the Contractor shall stop foundation preparation operations. The owner or inspector will review the foundation preparation requirements in view of the existing field information and subsequently direct the Contractor on any additional requirements for foundation preparation.

The stripped foundation beneath the impervious section of the embankment, including the key trench, shall be thoroughly scarified to a depth of 150 mm by diskng. This surface shall then be compacted to a density equivalent to that of the embankment. The foundation beneath the internal filter does not require scarification or compaction.

Materials For Embankment Construction

Embankment materials, impervious Zone 1 and random Zone 2 shall consist of low to medium plastic clay or a suitable homogeneous mixture of sand, gravel, silt or clay. This material should contain sufficient silt and clay fines to be relatively impermeable and suitable for compacting as specified. A zone downstream of an inclined or vertical filter may be constructed of random Zone 2 material as obtained from borrow sources without extra mixing other than that provided by standard excavating and spreading procedures. Embankment materials shall be free from vegetal growth, organic substances, stones greater than 150 mm diameter, ice, snow, and frozen materials.

Select filter sand Zone 3A material shall consist of clean, free draining, reasonably well-graded sand or sand and gravel mixtures having a maximum size of 75 mm and less than 4% silt or clay fines when in place.

Riprap bedding gravel Zone 3B shall consist of reasonably well-graded sand and gravel with at least 15% coarser than 38 mm and less than 8% silt or clay fines when in place.

Riprap Zone 4 and/or Zone 5 shall consist of hard, dense, durable fieldstone, cobbles or rock fragments but not sandstone, clay shale, or
clay ironstone. Flat slabby type rocks shall be excluded. Riprap shall be reasonably clean. Riprap shall have an average size (D50) and thickness as shown on the drawings. Riprap shall be reasonably well-graded with no sizes lacking and no excess of material in any size range.

**Moisture Requirements**

Impervious Zone 1 materials shall have a water content between 1% dry and 3% wet of optimum water content to obtain maximum density upon compaction. Random Zone 2 material shall have a water content ranging from 2% dry to 4% wet of optimum water content. Material with water contents between optimum and 4% wet of optimum water content shall be used along conduits, at the abutment contacts and adjacent to concrete structures to ensure intimate contact between soil and structure.

The water content of the fill material can be increased by sprinkling water on an uncompacted loose lift with a water truck or by irrigating the borrow pit. If sprinkling a loose lift with a water truck is employed, the lift shall then be thoroughly mixed by disk ing before compacting. Processing material on the fill may be difficult due to a relatively small working area. The difficulty increases when a large increase in water content is required. These difficulties may be avoided and construction expedited by adding water in advance at the borrow pit.

Ponding or irrigation with sprinklers in the borrow pit should begin at least two weeks prior to beginning construction and preferably longer as the depth of moisture penetration is dependent on the length of soaking time and soil type. When sufficient penetration has not been achieved prior to construction, the borrow pit could be divided into three sections of more or less equal area with one of these sections a- ways in the process of being wetted while the second is soaking and the third is being excavated. Drying of overly wet soils can be accomplished by disk ing the material and leaving it exposed to the atmosphere.

The water content of impervious materials in the borrow areas may change seasonally and should be confirmed just before construction. Allowance should be made for evaporation losses during excavating, hauling and placing. Losses in water content from evaporation are commonly in the range of 1% to 3% and may be as high as 5%or 6% depending on the weather.

**Placing Procedures**

Fill material shall be spread in lifts parallel to the embankment centreline to prevent a continuous stratified layer extending from up- stream to downstream. The lift thickness, before and after compaction, shall not exceed 250 mm and 150 mm respectively.

Fill materials shall not be placed in a frozen condition or on a frozen surface. Winter construction should be avoided as the density
obtained with a given compactive effort on impervious soil decreases noticeably when the temperature approaches the freezing point. However, granular materials may be placed in freezing conditions if the material is not placed in frozen lumps.

Granular materials may be placed in lifts of up to 300 mm thickness. Care shall be taken to prevent contamination of filter materials with clay and silt fines. The filter material may be end dumped from trucks at convenient locations and spread with a dozer or track loader. Care shall be taken to ensure that the filter exit is not covered with impervious material so that drainage can take place.

Riprap and bedding gravel may be placed by end-dumping from trucks and pushing the material into place. To avoid segregation which may occur with this method, tractors or winches may be used to lower trucks down the slope so that the rock can be dumped against previously placed rock. Alternatively, a front-end loader may be used from the bottom to travel over previously placed rock. In all placing methods some hand labour will be necessary to adjust surface rocks and to fill voids with smaller rock to produce a dense cover. Care shall be taken to insure that the specified thickness of riprap bedding exists under all riprap and that riprap placing methods do not displace the bedding gravel.

Compaction Requirements

Impervious fill shall be compacted to at least 95% of Standard Proctor maximum dry density. This can be achieved with 10 complete passes of a sheepsfoot roller having a unit foot pressure of at least 1700 kPa (250 psi) and preferably 2400 kPa (350 psi). The number of passes may be reduced to a minimum of 6 for random Zone 2 material. The foot length of the roller shall be at least 200 mm and the face area shall not be less than 4500 mm². The foot spacing shall provide approximately four (4) tamping feet for each one-quarter square metre of cylindrical surface.

Alternate compaction equipment may be used providing each compacted lift is scarified to a depth of at least 50 mm to prevent layering and at least 95% of Standard Proctor maximum dry density is obtained. Effort must be made to avoid uneven compaction and layering between lifts which may lead to subsequent problems of embankment settlement and seepage through the embankment to the downstream slope.

Compaction around conduits is critical since the conduit extends from the reservoir through the embankment to the downstream slope and seepage problems can easily develop. Backfill material shall be mechanically hand tamped within 900 mm of all structure and special care shall be taken to insure good compaction of all backfill beneath the haunches of circular conduits. Backfilling operations shall be conducted such that the fill is brought up evenly on each side of the structure or pipe to prevent unbalanced load on the structure. Equipment trackage can be utilized adjacent to conduits or concrete
structures outside of the 900 mm zone; however, caution must still be exercised to ensure that the conduit or concrete structure is not damaged or pushed laterally. Select filter sand Zone 3A around the conduit and in the conduit foundation shall be compacted with vibratory equipment.

Riprap bedding gravel may be spread on the upstream face of the embankment with a dozer or track loader and does not require additional compaction.

Measurement

"Excavation" will be measured for payment by the cubic metre determined by either:

(a) Surveys made prior to commencement of the excavation and the lines and grades shown on the drawings or

(b) Where no lines or grades are shown, surveys of the respective areas made prior to commencement of the excavation and after completion of the excavation.

"Compacted Embankment Zone 1 and/or Zone 2, Select Filter Sand Zone 3A, Bedding Gravel Zone 3B, Gravel Slope Protection Zone 3C, and Riprap Zone 4 and Zone 5" will each be measured for payment by the cubic metre determined by computation of the volume between (1) the foundation lines and grades where shown on the drawings or as determined on the basis of surveys made after embankment foundation excavation (stripping and key trench) except scarifying, and (2) the lines and grades of the completed structure as shown on the drawings or as modified by the contract authority.

Payment

"Excavation" including stripping, key trench and structure excavation, earth cut channel spillway excavation and borrow excavation will be paid for at the unit price per cubic metre bid therefor in the Unit Price Table. The bid price shall include the costs of excavating; loading, hauling, depositing in embankments, backfills, stockpiles or waste areas; and trimming and shaping of excavation, stockpile and waste areas.

"Compacted Embankment Zone 1A and/or Zone 2" will each be paid for at the applicable unit price per cubic metre bid therefor in the Unit Price Table. Such payments shall constitute full compensation for all work in connection with scarifying foundation and the spreading, drying if required, additional moisture if required, mixing, compacting, removing unsuitable material, rehandling embankment materials if necessary and all other incidental work required for the construction and protection of the embankment as specified. Payment will be made for this work regardless of the source of the placed materials, and will be in addition to any payment for excavating and transporting the material from necessary excavations or borrow areas.
"Select Filter Gravel Zone 3A, Bedding Gravel Zone 3B, Gravel Slope Protection Zone 3C and Riprap Zone 4 and Zone 5" will each be paid for at the applicable unit price per cubic metre bid therefor in the Unit Price Table. Such payments shall constitute full compensation for all work in connection with obtaining, processing, loading, hauling, depositing at point of use, spreading, compacting if required, and all other incidental work required for completion of this work as specified.
SPECIFICATION FOR CORRUGATED STEEL PIPE (CSP)

General

The work covered by this section includes the supply of all materials, labour, plant and equipment for the supply, fabrication and installation of all corrugated steel pipe and corrugated steel pipe products as specified or shown on the drawings.

Materials

Corrugated steel pipe materials shall conform to CSA CAN3-G401-M81. The corrugated steel pipe shall be galvanized and shall have a corrugation profile of 68 mm pitch, 13 mm depth for pipe sizes up to and including 1200 mm, and a corrugation profile of 125 mm pitch, 26 mm depth for pipe sizes greater than 1200 mm. The material sheet thickness for various pipe sizes shall be as shown on the drawings.

Helical pipe ends shall be re-corrugated to provide a minimum of two annular corrugations for coupling purposes.

Couplings shall be used on all field connections and consist of galvanized materials of the same thickness as the pipes to be connected. The couplings shall be Hugger Band H-13 (semi-corrugated bands) complete with "O-ring" gaskets (2 gaskets per coupler) as supplied by Armco or approved equal. Each coupling shall consist of a one piece or two piece coupler complete with attached brackets, bolts and nuts. In addition, two 13 mm diameter galvanized rods with silo-type lugs shall be included as part of the coupler. The 13 mm diameter rods shall be fitted into the corrugations of the coupler and when drawn together provide increased joint strength.

Fabrication

The fabrication of CSP products shall conform to CSA CAN3-G401-M81. Fabrication shall be performed in the shop and the finished product transported to the field.

Finished members shall be straight and true, and have no protrusions or irregularities at fabricated elbow, tee or stub sections.

All surfaces of welds and areas where the galvanized coating has been damaged during welding shall be cleaned by wire brushing by hand or power tools. The cleaned surfaces shall be resurfaced with Devcon Z or other approved protective coating conforming to CSA Standard CAN3-G401-M81. The Contractor shall contact the pipe supplier to determine the source and availability of repair coating.
Installation

Corrugated steel pipe products including pipes and fabricated items shall be placed according to the lines and grades shown on the drawings or as directed by the inspector or the owner. Installation procedures shall be according to these specifications and the manufacturer's recommendations. If the pipe is damaged during installation, the Contractor shall repair or replace damaged pipe sections to the satisfaction of the owner.

The foundation for a pipe shall consist of a uniformly graded excavated bed free from soft or vegetal material, rocks, and lumps of dried soil materials. Pipes shall not be placed on fill materials except in the area of the internal granular filter system and key trench. Compacted select filter sand (Zone 3A) and impervious compacted embankment (Zone 1) shall form the pipe foundation at these locations, respectively. If foundations for structures or pipes are not suitable for placement of the structure at the elevation specified, the Owner or inspector will determine the corrective action necessary for the work to proceed.

The pipe lengths shall be placed so that the interior seam or lap is on the downstream side of a corrugated sheet. In coupling pipe joints, the "O-ring" gaskets shall be placed on each pipe end at the first valley of the annular corrugation; the coupling band shall then be placed around the joint and the pipes positioned such that the corrugations of the coupler lineup with the second annular corrugation at each pipe end; the coupling shall then be drawn together with the attached brackets and bolts. The 13 mm diameter rods with silo type lugs shall be positioned on each corrugation of the coupler and drawn together; finally the bolts through the brackets attached to the coupler shall be re-tightened. The complete length of pipe conduit shall be placed on the excavated foundation, coupled, inspected and approved by the inspector or owner before commencement of backfilling.

The backfill material shall be compacted embankment and/or select filter sand as specified in the standard specification for "Earthwork". The backfill material shall be brought up evenly on both sides of the pipe to the level of the pipe crown along the entire conduit length. Care shall be taken to ensure that backfill under the pipe haunches is thoroughly tamped into intimate contact along the full corrugated surface of the exterior pipe wall. The backfill shall be compacted to achieve a density equivalent to that obtained in the embankment using mechanical hand tamping equipment to at least 300 mm above the crown of the pipe. Above this level, backfilling may be performed by machine, however, the material shall be rolled, not dropped into any excavation.

CSP fabricated items shall be adequately supported during all operations including coupling, backfilling and concreting.
Measurement

Corrugated steel pipe (straight pipe) will be measured for payment by the lineal meter in place from end to end along the invert of each segment of pipe but not including fabricated items. Corrugated steel pipe fabricated items will be considered as lump sum items.

Payment

Corrugated steel pipe sections and fabricated items will each be paid for at the applicable unit price per lineal meter or lump sum bid therefore in the Unit Price Table. The price shall include the costs of supply, fabricating, handling, placing and assembling the pipes including connecting hardware, couplings, gaskets, lubricants, coatings and all other items required to make a proper installation as shown on the drawings and specified herein. The cost of earthwork including excavation and backfill is paid for under a separate section.
SPECIFICATION FOR REINFORCED CONCRETE

General

The work covered in this section consists of furnishing all plant, labour and equipment, and performing all work necessary for supplying, transporting, storing and handling of cement, aggregates, concrete admixtures, water and reinforcing steel; batching, mixing, transporting, forming, placing, finishing, curing and protecting the cast-in-place reinforced concrete required for the completion of this work.

Reinforced concrete work in place shall conform to required lines, elevations, dimension and details shown on the drawings.

Materials

All materials for concrete construction shall conform to CSA Standard CAN3-A23.1-M90 "Concrete Materials and Methods of Concrete Construction."

Concrete shall be proportioned and supplied to meet the following requirements:

1) Cement: Type 50, Sulphate-Resistant.

2) Specified 28-day compressive strength: 30 MPa.

3) Class of exposure: F1, Table 7 of CSA A23.1-M90.

4) Nominal maximum size of coarse aggregate: 20 mm.

5) Slump at point of discharge: 80 + 20 mm.

6) Air Content: 4 to 7 percent.

7) A type WN, normal setting water-reducing strength-increasing admixture may be used by the Contractor.

8) Supplementary cementing materials or other admixtures shall require prior approval by the owner or inspector.

Reinforcement shall be deformed bars of Grade 300 or higher billet steel conforming to the applicable provisions of CSA:G30.12-M77.

Methods

The methods for proportioning, producing, forming, reinforcing, placing, consolidating, finishing, protecting and curing concrete shall be in accordance with CSA:CAN3-A23.1-M90.
All concrete forms and forming shall be straight, true and plumb; complete; and securely anchored before placement of concrete. Before a new placement is begun adjacent to any horizontal, vertical or other construction joint, the surface of the hardened concrete shall be cleaned of foreign matter and laitance. The surface cleaning shall be conducted in two phases. The first phase shall be performed approximately 10 to 15 hours after concrete is placed and shall entail scrubbing with wire brushes or wire brooms in order to remove laitance and expose the individual pebbles. The scrubbing shall be delayed for a few hours if the larger pebbles are loosened when the surface is scrubbed. The second phase of the surface cleanup shall be performed immediately before fresh concrete is placed on the hardened concrete. The surface shall be washed with water until the concrete is thoroughly cleaned of foreign matter and is thoroughly saturated. Directly before placing, free water shall be removed. The initial lift of concrete shall be not deeper than 300 mm and shall be thoroughly vibrated close to the joint to ensure complete contact of the mortar phase with the hardened concrete.

Sandblasting may be used in lieu of scrubbing for the first cleanup phase. This operation shall be performed on the hardened concrete prior to forming the next lift. Sandblasting shall be done with care and skill to remove the laitance film of mortar so that the coarse aggregate particles are exposed without undercutting the particles. After sandblasting, all loose particles shall be blown from the surface of the joint.

Concreting shall be performed in one continuous operation. Concrete shall be deposited in the forms as closely as practicable to its final position. Lateral movements of fresh concrete, which can cause segregation, shall be avoided. Concrete shall be placed in layers that are approximately horizontal. The depth of each layer shall be limited to ensure vibration into the previously placed layer. Concrete shall be confined in a suitable vertical drop pipe or trunk to within 1.5 m of the concrete in place.

Concrete when being placed shall be consolidated thoroughly and uniformly by means of hand tamping tools or vibrators to obtain a dense homogeneous structure free from cold joints, voids and honeycombs.

Concrete surfaces not bounded by forms shall be screeded and floated to provide a smooth dense surface finish. A special trowel or form shall be fabricated from sheet metal to provide corner or edge radiuses as shown on the drawings.

Freshly deposited concrete shall be protected from premature drying and extremes of temperature to ensure proper curing of the concrete. Moist curing with water damp materials, curing compounds or plastic sheets shall be applied to concrete surfaces for a minimum of 7 days at an ambient temperature above 10° C.
Measurement

Concrete will be measured for payment by the cubic meter satisfactorily placed in the work, determined by computation of the volume of concrete shown on the drawings or as modified by the owner. No deductions will be made for the volume of reinforcing steel and metal work embedded in the concrete.

Payment

Concrete will be paid for at the unit price per cubic meter bid therefore in the Unit Price Table. The price shall include the cost of furnishing all plant, labour, equipment, materials including cement and reinforcing steel required to complete the concrete work in accordance with the plans and specifications.
SPECIFICATION FOR MISCELLANEOUS METALWORK

General

The work covered in this section includes the supply of all materials, labour, plant and equipment for the supply, fabrication, and installation of all metalwork as specified or as shown on the drawings except corrugated steel pipes, reinforcing steel used in concrete and manufactured gates, valves, and accessories.

Materials

Metal materials shall be in accordance with the following table or as indicated on the drawings.

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
<th>Specification</th>
<th>Finish</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel Pipe</td>
<td>Black, welded, Type F standard weight</td>
<td>ASTM:A53-82</td>
<td>Painted</td>
</tr>
<tr>
<td>Guardrails</td>
<td>75 mm diameter</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Steel Pipe</td>
<td>Black, welded, Type F standard weight</td>
<td>ASTM:A53-82</td>
<td>Painted</td>
</tr>
<tr>
<td>Conduit</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Steel Chain</td>
<td>Class PC, 6 mm</td>
<td>ASTM:A413-80</td>
<td>Galvanized</td>
</tr>
<tr>
<td>Structural Steel</td>
<td>Plate, structural shapes</td>
<td>CSA:G40.21-M81</td>
<td>Painted</td>
</tr>
<tr>
<td>Steel</td>
<td>(Grade 300W)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bars and Rods</td>
<td></td>
<td>ASTM:A36M-87</td>
<td>Painted</td>
</tr>
<tr>
<td>Bolts, Washers</td>
<td>Hex Head</td>
<td>ASTM:A307-81</td>
<td>Galvanized</td>
</tr>
<tr>
<td>and Nuts</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Welding electrodes shall conform to the requirements of CSA:W48.1M1980.

Painting materials shall conform to the following Canadian Government Specifications Board (CGSB) and/or manufacturer's products number in accordance with the following:

(a) Primer - (CGSB:1-GP-48M "Primer, Marine, for Steel"

: #63048 Marine Primer by General Paint Ltd.

: #4570 Metal Primer by Glidden Paint Ltd.

(b) Marine Enamel Paint (CGSB:1-GP-61Ma "Enamel, Alkyd, Marine, Exterior and Interior")

:10-010 Marine Enamel by General Paint Ltd.

:22350 Super Marine Enamel by CIL Sherwin-Williams Paint
Steel shall be galvanized where shown on the drawings. Galvanizing shall be by the hot dip process in accordance with ASTM:A123-78.

**Fabrication**

All metalwork shall be fabricated in sections in the shop and transported to the site.

Structural steel fabrication shall be in accordance with the applicable portions of CSA:CAN3-S16.1-M78, Steel Structures for Buildings - Limit States Design, clause 26.

Finished members shall be free from kinks, bends, or winds. Dimensions shall be accurate within 2 mm.

Welding shall be by the electric-arc-welding process using a method which excludes the atmosphere from the molten metal and shall conform to CSA;W59-84 and AWS:D2.0-1966.

Unless otherwise specified, all deposited weld metal shall have mechanical properties not less than those specified for base metal welded and shall have chemical composition similar to that of base metal.

All exposed ferrous surfaces not embedded in concrete shall be cleaned in the shop immediately after fabrication and prior to the application of the first coat of paint according to industry good practise and paint manufacturer's recommendations.

All exposed surfaces of welds executed in the field during the erection and mounting of the steel assemblies shall be cleaned in the field immediately prior to touch-up painting.

All metalwork to be painted shall receive one coat of primer and two coats of marine enamel paint.

Paint that is loose, weakly bonded, blistered, abraded or otherwise defective shall be removed and the surface shall be cleaned, prepared and repainted in accordance with the manufacturer's recommendations.

Painting shall not be performed during damp weather or when temperature is below freezing. Subsequent coats shall not be applied until the preceding coats are thoroughly dry.

**Installation**

All metalwork shall be correctly aligned and installed true to the lines and grades shown on the drawings and so that proper matching of adjacent concrete surfaces shall be obtained.
All anchors and anchor bolts shall be installed so that after placement of the concrete, their position is within 3 mm of their theoretical location and within 2 mm of their theoretical location with respect to other anchors and anchor bolts which are to be subsequently connected to the same piece of metalwork.

All metalwork surfaces to be in contact with or embedded in concrete shall be thoroughly cleaned of all rust, dirt, grease, loose scale, grout, mortar or other foreign matter before installation.

Measurement

No measurement for Miscellaneous Metalwork is required.

Payment

Miscellaneous Metalwork will be paid for at the lump sum price bid therefore in the Unit Price Table. The price shall include all costs of furnishing, fabricating, galvanizing or painting where required and installing the materials as specified.
SPECIFICATION FOR GATE VALVES

General

The work covered in this section includes the supply of all materials, labour, plant and equipment for the supply and installation of gate valves as specified or shown on the drawings.

Materials

The gate valve shall be a Crane McAvity Resilient Seated Gate Valve complete with "flange" end and rubber gasket or approved equal.

The gate valve shall conform to ANSI/AWWA C509-80 "Resilient Seated Gate Valves, 3 through 12 NPS, for Water and Sewage System".

Miscellaneous valve stem extensions, support hardware, and fittings shall be as recommended by the manufacturer for buried service and as shown on the drawings.

Installation

Gate valves shall be installed according to the manufacturer's instructions.

Measurement

No measurement is required for Gate Valves.

Payment

Gate Valves will be paid for at the lump sum price bid therefore in the Unit Price Table. The price shall include all costs of furnishing and installing all materials as specified, including fasteners and hardware.
SPECIFICATION FOR POLYVINYL CHLORIDE (PVC) PIPE

General

The work covered in this section includes the supply of all materials, labour, plant and equipment for the supply and installation of PVC pipe as specified or shown on the drawings.

Materials

Polyvinyl Chloride (PVC) pipe shall be series 160, SDR 26 pipe as manufactured by IPEX or approved equal.

Polyvinyl Chloride (PVC) pipe described herein shall comply with the requirements of CSA/CAN3-B137.3-M86 "Rigid Poly (Vinyl Chloride) (PVC) Pipe for Pressure Applications".

The PVC pipe shall be homogeneous throughout, free from voids, cracks, inclusions and other defects, as uniform as practical in colour, density and other physical properties. Surfaces of the products shall be free from scratches, gouges and other imperfections.

PVC pipe shall be coupled with factory moulded bell ends or couplings with rubber ring joints. Flanged ends shall be installed at valve connections. PVC series pipe couplings and fittings shall be manufactured from CSA certified clean raw virgin material and the resin compound shall conform to ASTM:D1784-78, Class 12454-B and shall have a hydrostatic basis when tested and analysed by ASTM:D2837-76 of 27.58 MPa. Reworked material generated from the manufacturer's plant shall not be used to manufacture the pipe. The material shall be certified for potable water by the CSA Testing Laboratory.

Installation

Polyvinyl Chloride (PVC) pipe shall be installed to the lines and grades as shown on the drawings and according to the manufacturer's recommendation. The pipe shall be fully inserted into the bell end to accommodate potential movements.

The pipe shall be installed so that the barrel of the pipe is evenly supported throughout its entire length. The pipe shall be supported by a smooth, compacted bedding layer free from sharp projections, large dirt clods, stones greater than 40 mm in diameter or any frozen material. The backfill material shall be compacted in 100 mm layers to the specified density of the compacted embankment for the dam using mechanical hand tamping equipment to at least 300 mm above the top of the pipe to prevent undue pressures on the pipe. Above this zone, backfilling may be done by machines, however, the earth shall be rolled, not dropped into the excavations. All backfill shall be dry unfrozen material.
During all stages of construction, piping shall be protected from damage from any cause. Openings in the piping system shall be securely covered, capped or plugged to prevent collection of dirt, debris, or other extraneous matter during the entire construction. Damaged work shall be removed and replaced with new material.

Before the pipeline is backfilled it shall be pressure tested according to the manufacturers recommendations.

**Measurement**

Polyvinyl Chloride (PVC) Pipe will be measured for payment by the lineal meter in place measured horizontally from point of origin to end of run with no deductions for adaptors, valves, or joints.

**Payment**

Polyvinyl Chloride (PVC) Pipe will be paid for at the unit price per lineal meter bid therefore in the Unit Price Table. The price shall include all costs of supply, hauling, handling, placing, assembling and testing the pipe including jointing materials and testing equipment. The cost of earthwork including excavation and backfill is paid for under a separate section.
## Tender Form

**Joe Smith Dam**

<table>
<thead>
<tr>
<th>Item</th>
<th>Quantity</th>
<th>Unit of Measure</th>
<th>Unit Price</th>
<th>Extension</th>
</tr>
</thead>
<tbody>
<tr>
<td>Excavation</td>
<td>14000</td>
<td>m³</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Compacted Embankment (Zone 1)</td>
<td>12000</td>
<td>m³</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Select Filter Gravel (Zone 3A)</td>
<td>740</td>
<td>m³</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bedding Gravel (Zone 3B)</td>
<td>620</td>
<td>m³</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Riprap (Zone 4 and 5)</td>
<td>1240</td>
<td>m³</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete</td>
<td>12</td>
<td>m³</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pile Drilling</td>
<td>1</td>
<td>Lump Sum</td>
<td></td>
<td></td>
</tr>
<tr>
<td>100 PVC Pipe</td>
<td>47</td>
<td>m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>900 CSP Pipe</td>
<td>32</td>
<td>m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1800 CSP Riser Section (including 2000 and 2400 exterior sections)</td>
<td>1</td>
<td>Lump Sum</td>
<td></td>
<td></td>
</tr>
<tr>
<td>150 CSP End Sections (4 m long)</td>
<td>1</td>
<td>Lump Sum</td>
<td></td>
<td></td>
</tr>
<tr>
<td>100 Gate Valves</td>
<td>1</td>
<td>Lump Sum</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Total**
APPENDIX E - DEFINITIONS
Borrow Area

The borrow area is a specified area owned or controlled by the project proponent which provides a source of soil materials for embankment construction.

Consolidation

Consolidation of embankment is a process where a soil mass compresses, densifies and expels air and water from void spaces due to compactive effort, additional loading and/or to the self weight of the soil mass.

Consolidation of concrete is a process of compacting fresh concrete to mould it within forms and around embedded materials or reinforcement and to eliminate entrapped air which causes honeycomb.

Demand

A demand is a claim or requirement for water. The demand may not be completely satisfied during a low runoff period or when the reservoir storage is depleted.

Design Life

The design life of a structure or project is the estimated time or duration in which, with normal maintenance, the structure or project can be operated before it requires major repair or replacement.

Dilatancy

Dilatancy is the reaction to shaking. A fine-grained non-plastic soil will show free water on the surface while being shaken. Squeezing or remolding will cause the water to disappear from the surface.

Draft

A draft can be considered as a completely or partially satisfied demand. The firm annual draft is the demand that can be completely satisfied each year of the study period. The expression "draft with shortages" implies that the demand cannot be satisfied all of the time.

For example, the expression draft with shortages in 30% of the years means that in 70% of the years, the demand will be completely satisfied, but in 30% of the years, the project will experience some degree of shortage. A user of water that operates under the restriction of having shortages in 30% of years must either accept shortages, impose rationing, or look to other sources (e.g. groundwater) to help meet the demand.

Erratic

A rock fragment carried by glacial ice and deposited when the ice melted at some distance from the outcrop from which the fragment was
derived generally of boulder size, although the fragments range from pebbles to house-sized blocks.

Evaporation from Reservoirs

The terms gross evaporation, precipitation, net evaporation, as they apply to reservoir evaporation, are explained as follows:

a) **Gross Evaporation** is the potential evaporation (calculated as depth) from a free water surface.

b) **Precipitation** is the depth of water (i.e. water equivalent of all forms of precipitation) that falls on the reservoir surface.

c) **Net Evaporation** is the resultant net change in depth of a water surface based on the potential evaporation (gross evaporation) and the precipitation (i.e. Net Evaporation = Gross Evaporation - Precipitation).

Finger Drain

A finger drain is a portion of free-draining filter material of specified cross-section and length which acts as a passage for transporting seepage collected from a vertical and horizontal filter through the downstream portion of the dam embankment.

Flood Hydrograph

A graph showing the relationship of flowrate with respect to time.

a) **Recorded Hydrograph** - A graph of a recorded event.

b) **Design Hydrograph** - A graph of a "designed" event as conceived by a hydrologist for a specific function (e.g. spillway design).

Flood Surcharge Elevation

The flood surcharge elevation is the reservoir elevation required to provide the head or energy for passing the flood flows through the spillway system.

Frequency Analysis

A statistical procedure for estimating the frequency of occurrence of past events and for estimating the probability of occurrence of future events, including events of greater magnitude than those which have occurred in the past.

Full Supply Level (FSL)

The full supply level of a reservoir impounded by a dam is the water surface elevation below which all inflows may be stored (if outlet
gates are closed) and above which all inflows will be passed through a spillway system. It is the maximum level at which the dam is designed to store water for extended periods of time.

**Hazard Potential**

The hazard potential of a dam is a relative measure of the potential loss with respect to life, flood damage and lost benefits from a hypothetical failure of the dam.

**Hole Log**

A hole log provides a description (qualitative and quantitative) of the soil stratigraphy at a specific location based on drilling or excavated test pits.

**Homogeneous**

A homogeneous material is composed of the same constituents and is uniform throughout its structure or composition.

**Instantaneous Flood Peak**

The maximum discharge rate over an extremely short duration of time. Applies to both a recorded hydrograph and a design hydrograph.

**Live Storage**

Live storage is the volume of water available for utilization between the FSL and the invert of the device used to obtain the water from the reservoir.

**Median Annual Runoff (MAR)**

The median annual runoff at a site is a measure of the annual runoff to the site and is the product of the effective drainage area and the median annual unit runoff.

**Median Annual Unit Runoff (MAUR)**

The median annual unit runoff in \( \text{dam}^3/\text{km}^2 \) is a measure of the "average" annual runoff for a given region. The middle value in an ordered array of runoff volumes (i.e. median) is used as the indicator of average runoff rather than the arithmetic mean. (The arithmetic mean could be unduly influenced by extreme high or low values).

**Operating Spillway Design Flood (OSDF)**

The operating spillway design flood is the flood flow selected for sizing the operating spillway such that no damage occurs to the structure or embankment.
Pore Water

Pore water is the water in the void spaces in a soil mass.

Reservoir Storage

The term "storage" in this manual is synonymous with the term "reservoir storage" and means the total volume of water stored at a given elevation.

Safety of Dam Design Flood (SDDF)

The Safety of Dam Design Flood is the maximum flood which could be passed by the spillway system without overtopping the embankments while maintaining a small nominal freeboard between the maximum reservoir level attained during passage of that flood and the top of the embankment. This nominal freeboard allowance provides a measure of safety against dam overtopping due to the methods employed for analysis in this manual and the unknowns associated with statistical quantification of natural phenomena.

The freeboard allowance is not intended to provide a safety factor to account for inaccuracies in the calculations or errors in judgement made by the designer.

Storage Dam

A storage dam is an earth embankment located in a natural water course which captures or retains water from seasonal precipitation events for later use. The embankment structure may retain water continually over a period of years and cause saturation of the foundation and embankment materials coincident with design criteria and assumptions.
APPENDIX F

UPDATED HYDROLOGY SECTION
APPENDIX F - UPDATED HYDROLOGY SECTION

This appendix supersedes the Hydrology Section (Section 2.4) presented in the main manual on pages 2-29 to 2-62. The methodologies described in this appendix are significantly different from those presented in Section 2.4. This update is based on a larger data base (e.g., flood potential based on 440 station-years of data as compared to 250 station-years), pertinent regional assessments (e.g., water supply potential based on recently published Annual Unit Runoff and Historical Low Annual Runoff Studies) and less subjectivity. Use of the improved methodologies within this appendix for small dams (as defined in the Manual) is highly recommended. However, to assess water supply potential, consultation of a hydrologist may be required to run the Hydrology Division’s HY01, HY02 and HY03 computer programs.

Revision of the Hydrology Section was warranted considering the availability of 32 years of flood peak data provided by the Swift Current Research Station for four very small plots ranging in size from 4.25 ha to 5.06 ha and for a larger plot of 58 ha. During this reassessment, it was decided to revise the original flood potential methodology using the Swift Current information along with several years of additional data collected at the other previously-used gauging stations. At the same time, a procedure had been developed to estimate critical low flow sequences for projects with small contributing drainage basins. Hence, the water supply potential methodology was also revised. The resulting updated procedures are much less subjective than the original procedures.

Additional definitions and references specific to the Updated Hydrology Section are included at the end of the appendix. Revised “Hydrology Summary Report” sheets are also included.

The Updated Hydrology Section is an improvement to, and is recommended over, those methods presented in the original manual (1992) in terms of less subjectivity, larger data base and pertinent regional assessments.
# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>F-1 GENERAL</td>
<td></td>
</tr>
<tr>
<td>F-2 FLOOD POTENTIAL</td>
<td></td>
</tr>
<tr>
<td>F2A General Concepts</td>
<td>F2</td>
</tr>
<tr>
<td>F2B Development of Flood ...</td>
<td>F2</td>
</tr>
<tr>
<td>F2C Procedures for ...</td>
<td>F3</td>
</tr>
<tr>
<td>F2D Example of Flood Peak ...</td>
<td>F3</td>
</tr>
<tr>
<td>F2E Uses and Limitations</td>
<td>F4</td>
</tr>
<tr>
<td>F-3 WATER SUPPLY POTENTIAL</td>
<td></td>
</tr>
<tr>
<td>F3A General Concepts</td>
<td>F4</td>
</tr>
<tr>
<td>F3B Development of Water Supply</td>
<td></td>
</tr>
<tr>
<td>Determination Procedures</td>
<td>F6</td>
</tr>
<tr>
<td>F3C Procedures for Water Supply</td>
<td></td>
</tr>
<tr>
<td>Potential Determination</td>
<td>F7</td>
</tr>
<tr>
<td>F3D Examples of Water Supply</td>
<td></td>
</tr>
<tr>
<td>Potential Determination</td>
<td>F8</td>
</tr>
<tr>
<td>F3E Uses and Limitations</td>
<td>F11</td>
</tr>
<tr>
<td>F-4 HYDROLOGY SUMMARY REPORT</td>
<td>F12</td>
</tr>
<tr>
<td>F-5 DEFINITIONS</td>
<td>F32</td>
</tr>
<tr>
<td>F-6 REFERENCES</td>
<td>F33</td>
</tr>
</tbody>
</table>
LIST OF TABLES

Table
   No.

F.1  Monthly Distribution of Gross Evaporation and Precipitation in the Canadian Prairies

LIST OF FIGURES

Figure
   No.

F.1  Flood Determination Equations
F.2  Annual Unit Runoff (dam³/km²) for a 50% (Median) Probability of Exceedence
F.3  Annual Unit Runoff (dam³/km²) for a 10% Probability of Exceedence
F.4  Annual Unit Runoff (dam³/km²) for a 25% Probability of Exceedence
F.5  Annual Unit Runoff (dam³/km²) for a 70% Probability of Exceedence
F.6  Annual Unit Runoff (dam³/km²) for a 75% Probability of Exceedence
F.7  Annual Unit Runoff (dam³/km²) for a 80% Probability of Exceedence
F.8  Annual Unit Runoff (dam³/km²) for a 90% Probability of Exceedence
F.9  Maximum Number of Consecutive Years in the 43-Year Period 1952-92 that the Annual Runoff Volume was Zero
F.10 Maximum Number of Consecutive Years in the 43-Year Period 1950-92 that the Annual Runoff Volume was Less Than 10% of the Median
F.11 Maximum Number of Consecutive Years in the 43-Year Period 1950-92 that the Annual Runoff Volume was Less Than 25% of the Median
F.12 Maximum Number of Consecutive Years in the 43-Year Period 1950-92 that the Annual Runoff Volume was Less Than 50% of the Median
F.13 Maximum Number of Consecutive Years in the 43-Year Period 1950-92 that the Annual Runoff Volume was Less Than 75% of the Median
F.14 Maximum Number of Consecutive Years in the 43-Year Period 1950-92 that the Annual Runoff Volume was Less Than the Median
F.15 Mean Annual Gross Evaporation (mm) in the Prairies for the 30-Year Period 1961-90
F.16 Mean Annual Precipitation (mm) on the Prairies for the 30-Year Period 1961-90
F.17 Mean Annual Ice Thickness for Small Prairie Water Bodies
F.18 Hydrology Summary Report
F-1  GENERAL

The water supply potential is required in the early planning stage to determine if the watershed is able to supply sufficient water to satisfy project demands. The flood potential is required to size the spillway components. The project designer conducts the hydrology studies to determine the water supply potential and flood potential and summarizes the information in a standard "Hydrology Summary Report".

This appendix provides a convenient handbook method for estimating flood peaks and evaluating water supply potential for small prairie watersheds (i.e. 50 km² or less). Hydrological information for larger watersheds can be obtained from a qualified professional. This appendix was prepared with the following attributes in mind:

1. The handbook 'section' (instructions) should be easy to use and should provide quick but appropriate estimates.

2. The assumptions and engineering judgements made in preparing the handbook 'section' should be logical and reasonable.

3. The judgment required by the user should be minimized.

This appendix has an "explanation" section and a "how to do it" section for both the Flood Potential Subsection and the Water Supply Potential Subsection. The user should read the explanation for each of these subsections so as to understand the relevant hydrological concepts. However, when assessing the hydrologic aspects of a project, the prescribed procedures can be used much like a handbook, with a minimum amount of judgement.

An important hydrological concept that pertains to both flood and water supply potential is the concept of drainage areas. For many prairie streams, the drainage area contributing to runoff is not constant for all runoff events, but fluctuates from one event to another. From this general concept of fluctuating drainage area, two arbitrary concepts were proposed by Stichelng and Blackwell. These concepts are described below.

The gross drainage area of a stream at a specified location is that area encompassed by its height of land boundary (i.e. the drainage divide between adjoining watersheds).

The effective drainage area is that portion of a drainage area that might be expected to contribute to runoff in an average year. This area excludes marshes, sloughs and lakes that would not spill to the main stream in an average year, along with their local drainage areas.

In theory, gross and effective drainage area boundaries would appear to be distinct. However, in drawing a line on a topographic map that represents a drainage area boundary, considerable judgement is required, especially for poorly-drained watersheds which are characteristic of the prairies. The gross drainage area is bounded by the height of land of the watershed and, in theory at least, the gross
drainage area boundary is a definite line on the earth defined primarily in terms of topography. By contrast, the effective drainage area boundary is a conceptual line that encloses an area which contributes to runoff under "average" conditions. Thus, the effective drainage area boundary is somewhat subjective and is defined primarily in terms of hydrologic factors.

A description of other hydrological terms are included in Appendix E - "Definitions" or in this appendix on Page F32.

F-2 FLOOD POTENTIAL

F2A General Concepts

Ideally, the estimate of the magnitude of flood peaks and their probability of occurrence should be based on a statistical analysis of recorded streamflows at or near the project site. Unfortunately, there are very few small watersheds that have records, and those that do have records do not have a sufficient period of record to adequately define the 1:100 or even the 1:50 flood. Thus, attempts to estimate flood peaks must, of necessity, be inferred from the limited hydrological data available using mathematical models of varying degrees of sophistication and/or complexity.

The approach proposed in this appendix was based on the observed flood peaks for snowmelt and rainfall runoff on a number of small streams in Saskatchewan. For a given probability of occurrence (i.e. 1:2 event), the magnitude of the flood peak will depend solely on the effective drainage area of the basin.

Since the magnitude of the flood peak is the primary concern, drainage areas that are controlled by structures that significantly restrict or detain peak flows should be excluded from the effective drainage area. The restrictive structures may include dams, railway crossings, road crossings, etc. Field inspection may be necessary as some structures (culverts, etc.) may be designed to only slightly restrict flows during rare events.

F2B Development of Flood Determination Equations

Twenty-one flow-metering sites (all in Saskatchewan) were considered suitable (small drainage basins and sufficient data) as a basis for estimating the flood peaks for small basins. Of the 21 basins investigated, seven were Water Survey of Canada (WSC) stations, nine were PFRA Spring Runoff Monitoring Program stations and five were Agriculture Canada Swift Current Research Branch stations. Only stations having at least ten years of recorded annual (spring or summer) instantaneous peak flows were selected. The average length of record was 21 years. For each station, the annual instantaneous flood peaks were ranked, assigned Cunnane plotting positions, and plotted on logarithmic probability paper. A best fit line drawn "by eye" was fitted through the data points and the 1:2, 1:5, 1:10, 1:20, 1:50 and 1:100 instantaneous flood peaks were extracted for each station.
Multiple regression analyses were conducted relating flood peaks (ranging from 1:2 to 1:100) to various physiographic basin parameters such as: effective drainage area (with values ranging from 4.25 ha to 46.1 km²), slope (ranging from 0.0019 m/m to 0.0346 m/m), length of basin (ranging from 0.115 km to 21.8 km), a basin shape factor determined as effective drainage area/basin length (ranging from 0.34 km to 4.28 km), etc. For all flood peaks (1:2 to 1:100 events), the most significant simple (one independent variable) correlations were obtained with the logarithmic values of flood peak related to the logarithmic values of effective drainage area. The resulting regression equations had high correlation coefficients, ranging from 0.928 to 0.961. Incorporating additional parameters into the equations did not significantly increase the statistical confidence of the relationships. As a result, and in order to keep the equations as simple as possible, the one-parameter effective drainage area equations were selected. The resulting equations for the various return periods are shown on Figure F.1 and are summarized as follows:

\[
\begin{align*}
\log Q_{2} & = 0.68879 \log (EDA) - 0.79512 \\
\log Q_{5} & = 0.70957 \log (EDA) - 0.33964 \\
\log Q_{10} & = 0.71559 \log (EDA) - 0.12846 \\
\log Q_{20} & = 0.71711 \log (EDA) + 0.03746 \\
\log Q_{50} & = 0.71863 \log (EDA) + 0.21923 \\
\log Q_{100} & = 0.71852 \log (EDA) + 0.33504 
\end{align*}
\]

where: \( Q_{n} \) = 1:2 maximum annual instantaneous flood peak (m³/s) \\
EDA = effective drainage area (km²)

F2C Procedures for Flood Peak Determination

a) Delineate Effective Drainage Area

Delineate the effective drainage area tributary to the study site using procedures recommended by Mowchenko and Meid. Make a field inspection of the study basin, if necessary, to help evaluate the drainage boundary. Exclude drainage areas that are controlled by structures that significantly restrict or detain peak flows.

b) Determine Flood Peak

Using the effective drainage area determined in a), calculate the required flood peak(s) using the flood determination equations presented in Section F2B or Figure F.1.

F2D Example of Flood Peak Calculation

The following example illustrates the procedure required to calculate flood peaks for a given project. The project has an effective drainage area of 20 km² as determined from 1:50,000-scale NTS maps and
field inspection. The slope of the watershed is 0.002 m/m. Using the
equations from Section F2B or Figure F.1 the resulting peak flows are:

\[
\begin{align*}
Q_2 &= 1.3 \text{ m}^3/\text{s} \\
Q_5 &= 3.8 \text{ m}^3/\text{s} \\
Q_{10} &= 6.3 \text{ m}^3/\text{s} \\
Q_{20} &= 9.3 \text{ m}^3/\text{s} \\
Q_{50} &= 14 \text{ m}^3/\text{s} \\
Q_{100} &= 19 \text{ m}^3/\text{s}
\end{align*}
\]

F2E Uses and Limitations

The equations apply to small prairie watersheds (i.e. 50 km² or
less, with watershed slopes less than 0.035 m/m) whose natural flow
regime has not been appreciably altered. If a watershed is steeper than
0.035 m/m or contains dams, embankments or drains that could appreciably
alter normal flows, appropriate adjustments to design floods should be
made. These cases should be referred to a qualified professional for
assistance.

F-3 WATER SUPPLY POTENTIAL

F3A General Concepts

The water supply potential of a basin depends on the runoff volume
from the basin and the ability of the user to utilize this runoff. Better control and utilization of the runoff can be obtained through use of
a reservoir to carry over runoff water from month to month and from
year to year. However, in some instances, runoff water will be diverted
directly from the live stream and used for such purposes as backflood
irrigation (i.e. without the benefit of a storage reservoir). This
manual provides guidelines for evaluating the water supply potential of
a basin for both storage and non-storage conditions.

a) Storage Condition (i.e. Reservoir)

Assuming that a storage reservoir is used, the water supply
potential of a watershed depends primarily on the following seven
factors.

1. Magnitude of Runoff Volume

The median annual runoff at a reservoir site is commonly used as
an indicator of basin runoff potential. In the report entitled
"Annual Unit Runoff on the Canadian Prairies", the Hydrology
Division has developed a map showing the median annual unit runoff
for the Canadian Prairie region. Refer to Figure F.2. The report
also provides maps showing the annual unit runoff for a number of
other levels of probability of exceedence. Refer to Figures F.3
to F.8.
2. Runoff Pattern

The variability of runoff volumes from one year to the next, as evidenced by the number of consecutive low flow years, affects the draft that can be obtained from a reservoir. The Hydrology Division, in a report entitled "Historical Low Annual Runoff for the Canadian Prairies", has developed a procedure to determine the most critical low flow sequence that could have occurred in a region during the 43-year period 1950-92. This procedure is based on Figures F.9 to F.14 which show the maximum number of consecutive years, plotted at the centroid of the effective drainage basin contributing to the specific hydrometric gauging station (229 stations), that the annual runoff was less than a specified percentage of the median.

3. Reservoir Capacity

The magnitude of the reservoir capacity in relation to the annual runoff volumes has a major impact on the available draft.

4. Reservoir Efficiency

Because evaporation losses decrease the available draft of a reservoir, an efficient reservoir will have a small water surface area relative to the volume of water stored. Thus, reservoir efficiency increases as the ratio of surface area to volume of water stored decreases.

5. Net Evaporation

Net evaporation varies throughout the Canadian Prairie region. As the evaporation rate increases, the amount of stored water that is available for use decreases. Net evaporation is calculated as gross evaporation minus precipitation. Mean annual gross evaporation, based on the standard 30-year period 1961-90, for any location on the Canadian Prairies can be determined from Figure F.15. Gross evaporation was determined in the Hydrology Division report entitled "Gross Evaporation for the 30-Year Period 1961-90 in the Canadian Prairies". Likewise, for the same standard period, mean annual precipitation can be determined from Figure F.16. The monthly distributions of both gross evaporation and precipitation are provided in Table F.1.

6. Reservoir Withdrawal Pattern

Because winter ice decreases the amount of stored water that is available for use during the winter, a summer withdrawal pattern will provide a larger draft than a year-round withdrawal pattern. A variable withdrawal pattern will also provide a somewhat different draft than a uniform withdrawal pattern.

7. Acceptability of Shortages

Accepting shortages in some of the years allows for a greater utilization of stored water in the long run. As more shortages are
accepted, more of the stored water is used when it is available. Thus, less water is lost to evaporation and spillage because less water is carried over from year to year.

From a hydrologic perspective, the optimum storage of a reservoir is that storage beyond which an increase in storage does not produce an appreciable increase in draft. This optimum storage can be determined from a storage-draft curve and is that point in the curve where the rate of change is at a maximum. There is usually not a sharp break in the curve so that the optimum storage is selected based on judgment. A "rule of thumb" for optimum storage would suggest that the capacity of the reservoir should be three to four times the median annual runoff. The optimum storage will vary depending on the demand pattern placed on the reservoir. Furthermore, the optimum storage for a firm annual draft condition will be different from that for a draft with an allowable shortage condition. The optimum storage from a hydrologic perspective may be different than the optimum storage from an economic perspective.

b) **Non-Storage Condition (i.e. Utilize Live Stream)**

If withdrawal is from the live stream only, as would be the case for a spring backflow project, the water supply potential depends solely on the magnitude and variability of runoff from year to year. In this case, only a small withdrawal if any, can be expected on a firm basis (i.e. guaranteed annual diversion). Thus, a "reasonable" diversion can be expected in only a percentage of the years (i.e. not every year). The magnitude of withdrawal that is possible 70% of the time (refer to Figure F.5) is considerably less than that which is possible 50% of the time (refer to Figure F.2). (The magnitude of the annual volume that is divertible 100% of the time would be zero for streams which may experience even one year of zero flow.)

F3B **Development of Water Supply Potential Determination Procedure**

The following information describes the background philosophy and investigative methods used in developing the procedures for estimating the project water supply potential. These procedures allow the estimation of the water supply potential of a basin for both storage and non-storage conditions.

a) **Storage Condition (i.e. Reservoir)**

Ideally, the water supply potential of a storage project should be determined using a water supply model whose inputs would include a storage-area relationship, historic monthly inflows and net evaporation over a long period of time (50+ years), and specified demands (both magnitude and distribution). However, the basic historic data are not readily accessible to field offices. Thus, an approach that would be appropriate to field office facilities was developed.

The developed approach is one that will provide an estimate of the water supply potential of a project based solely on the most critical low flow sequence that could have occurred over the 43-year period 1950-92, determined from the median annual runoff volume and the maps of
consecutive years of low runoff (see Figures F.9 to F.14). The median annual runoff to the reservoir can be determined by using the median annual unit runoff map for the Canadian Prairie region (Figure F.2) and the effective drainage area tributary to the reservoir site as determined by the field office. The resulting low flow sequences can be used to design reservoirs or to assess the performance of existing projects.

Comparison of the annual runoff volumes during the critical low runoff sequence with the project size and demand may ascertain whether the project will or will not meet the demands. In cases where uncertainty exists, a water budget program such as the Hydrology Division's HY02 Project Sizing Program can be used to determine the most appropriate size of the project for a given demand, or the Hydrology Division's HY03 Single Reservoir Simulation Program may be used to assess water supply with respect to potential low runoff and project size.

b) **Non-Storage Condition (i.e. Utilize Live Stream)**

Maps found in the "Annual Unit Runoff on the Canadian Prairies" report can be used for non-storage condition analyses. These maps (see Figures F.2 to F.8) can be used for any location within the Canadian Prairie region to determine the annual runoff volumes that would be equalled or exceeded 10%, 25%, 50% (median), 70%, 75%, 80% and 90% of the time.

**F3C Procedures for Water Supply Potential Determination**

The procedures for determining the water supply potential for both storage and non-storage conditions are described as follows.

a) **Storage Condition (i.e. Reservoir)**

This procedure can be used to determine either the available draft that can be obtained from a given storage or the storage required to provide a given demand. The following steps are common to both of these analyses.

1. Delineate the effective drainage area tributary to the damsite or study site on 1:50,000-scale MTS maps, making a field inspection as required.

2. Determine the median annual unit runoff (MAUR) at the site location from Figure F.2.

3. Calculate the median annual runoff (MAR) to the project or study site as follows:

   \[ MAR = MAUR \times \text{Effective Drainage Area} \]

4. Extract the maximum number of consecutive years of low runoff for each threshold from Figures F.9 to F.14 using the closest hydrometric gauging station(s).
5. From the resulting maximum number of consecutive years for each threshold (may be an average if more than one station is selected), determine the magnitude of the annual runoff in the low runoff sequence. Calculate the annual runoff volume for each range at the midpoint of the range (see Storage Project example in subsection F3D).

6. Derive the critical low annual runoff sequence by ranking the resulting annual runoff volumes from the highest to lowest and bounding this sequence by the median annual runoff volume.

7. Conduct a water supply assessment based on the critical low runoff sequence.

b) Non-Storage Condition (i.e. Utilize Live Stream)

The following procedure is used to determine the water supply potential at a point of interest (e.g. diversion point).

1. Delineate the effective drainage area tributary to the damsite or study site on 1:50,000-scale NTS maps, making a field inspection as required.

2. From Figures F.2 to F.8 determine the annual unit runoff for the desired probability of exceedence (i.e. 10%, 25%, 50%, 75%, 80% or 90% probability of exceedence).

3. Multiply the annual unit runoff for the desired probability of exceedence by the effective drainage area to determine the runoff volume in dam³. Note that this procedure assumes that diversion facilities are adequate to fully utilize the available water.

F3D  Examples of Water Supply Potential Determination

Examples for two types of projects (i.e. storage and non-storage) are provided to illustrate the use of the indicated procedures.

a) Storage Project

A water supply investigation utilizing a storage reservoir will normally have one of the following two objectives:

1. to size the reservoir to meet a demand;

2. to determine the available draft for a given reservoir size.

Note that for a given watershed there are practical hydrological limits to reservoir size and available draft.

The following example illustrates the procedure required to estimate the water supply potential for a proposed or existing storage site located near Dauphin, Manitoba. Given that there are no other users upstream or downstream of the storage site and that
the effective drainage area is 2.6 km², the critical low annual runoff sequence to the site is determined as follows.

1. The effective drainage area is 2.6 km² (given).

2. Using Figure F.2, the median annual unit runoff (MAUR) for the site is determined to be 75 dam³/km².

3. The median annual runoff (MAR) is calculated as follows:

   \[
   \text{MAR} = \text{MAUR} \times \text{Effective Drainage Area} \\
   = 75 \times 2.6 \\
   = 195 \text{ dam}^3.
   \]

4. From inspection of any one of the Historical Low Annual Runoff figures (Figures F.9 to F.14), the closest hydrometric gauging station to the project site has an effective drainage area centroid immediately south of Dauphin. From Figures F.9 to F.14, the maximum number of consecutive years of low annual runoff are read and are presented in the following table.

**Maximum Number of Consecutive Years of Low Annual Runoff near Dauphin, Manitoba**

<table>
<thead>
<tr>
<th>Range of Low Annual Runoff Volumes</th>
<th>Maximum Number of Consecutive Years</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zero</td>
<td>0</td>
</tr>
<tr>
<td>&lt; 10% of the median</td>
<td>0</td>
</tr>
<tr>
<td>&lt; 25% of the median</td>
<td>1</td>
</tr>
<tr>
<td>&lt; 50% of the median</td>
<td>2</td>
</tr>
<tr>
<td>&lt; 75% of the median</td>
<td>3</td>
</tr>
<tr>
<td>&lt; the median</td>
<td>4</td>
</tr>
</tbody>
</table>

5. From the above table and the median annual runoff volume of 195 dam³, runoff volumes are determined assuming that the annual runoff volume for each range is at the midpoint of that range. The results are presented in the following table.
Calculation of Critical Low Annual Runoff Volumes at Site near Dauphin, Manitoba

<table>
<thead>
<tr>
<th>Incremental Range of Low Annual Runoff Volumes (% of Median)</th>
<th>Midpoint of Annual Runoff Volume Range (% of Median)</th>
<th>Annual Runoff Volume to Project Site (dam³)</th>
<th>Number of Years</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.0</td>
<td>0.0</td>
<td>0</td>
</tr>
<tr>
<td>0 - 10</td>
<td>5.0</td>
<td>9.8</td>
<td>0</td>
</tr>
<tr>
<td>10 - 25</td>
<td>17.5</td>
<td>34</td>
<td>1</td>
</tr>
<tr>
<td>25 - 50</td>
<td>37.5</td>
<td>73*</td>
<td>1**</td>
</tr>
<tr>
<td>50 - 75</td>
<td>62.5</td>
<td>122</td>
<td>1</td>
</tr>
<tr>
<td>75 - 100</td>
<td>87.5</td>
<td>171</td>
<td>1</td>
</tr>
</tbody>
</table>

* (e.g. 37.5% of 195 dam³ = 73 dam³)
** (e.g. from the Maximum Number of Consecutive Years table in Step 4, two years are <50% of the median and one year is <25% of the median, thus one year is in the 25-50% range)

6. The critical low runoff sequence is determined from the information provided in the previous step. The years immediately preceding and following the critical four-year period must have had runoff equal to or greater than the median annual runoff of 195 dam³, since the table (4) indicates that there was only a maximum of four consecutive years when the annual runoff volume was less than the median annual runoff volume. Assume that these two years had runoff equal to the median and rank the other annual runoff volumes from highest to lowest (generally the most critical sequence for surface water supply potential assessment). The resulting critical low annual runoff sequence is:

Critical Low Annual Runoff Sequence

<table>
<thead>
<tr>
<th>Year 1</th>
<th>Year 2</th>
<th>Year 3</th>
<th>Year 4</th>
<th>Year 5</th>
<th>Year 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>195 dam³</td>
<td>171 dam³</td>
<td>122 dam³</td>
<td>73 dam³</td>
<td>34 dam³</td>
<td>195 dam³</td>
</tr>
</tbody>
</table>

7. The water supply potential is assessed using either simulation or comparative techniques. With the resulting critical low annual runoff sequence, Hydrology Division's HY02 Project Sizing Program can be used to size a reservoir for a given annual demand. To simplify the analysis, the annual runoff for each year could be assumed to occur during one spring month (e.g. April) when runoff in the area normally occurs. If it is not possible to increase the reservoir capacity or annual runoff volumes (e.g. snowtrapping, drainage area enlargement) to eliminate shortages, Hydrology Division's HY03 Single Reservoir Simulation Program can be used to determine
the number of years that shortages would have occurred and the magnitude of those shortages during the critical low runoff sequence. To determine the firm draft from a reservoir, Hydrology Division's HY01 Water Supply Potential Program can be used with the same critical low annual runoff sequence. In this example, if the demand on the reservoir is small, say less than 5 dam³, the critical low runoff sequence (with the lowest annual runoff of 34 dam³) would have no bearing on the project size and the project could reliably be designed for a one year supply plus evaporation and ice formation (if winter withdrawals are required).

To run the models described in (7), the following additional information is required: elevation-storage-flooded area relationship, dead storage, net evaporation, and ice depth if winter withdrawal is considered. Net evaporation is determined from the at-site mean annual gross evaporation (obtained from Figure F.15) and mean annual precipitation (obtained from Figure F.16). The monthly net evaporation is then determined by first distributing the annual gross evaporation and precipitation into monthly values using Table F.1, and then subtracting monthly precipitation from corresponding monthly gross evaporation. Ice depth can be obtained from Figure F.17, which was extracted from the Hydrology Division's report entitled "Determination of Ice Thickness for Small Water Bodies in the Canadian Prairie Region".

b) Diversion Project (i.e. no storage)

An assessment of the backflood potential is required at the same location as in the previous example (a). Assume that diversion is from a live stream and the capacity of the diversion facilities (structure and canal) will not limit the desired divertible flow. The intent is to provide a volume of water which is available in seven out of ten years (70% probability of exceedence). The procedure required to determine the water supply potential for diverting from a live stream is illustrated by the following example.

1. The effective drainage area is 2.6 km² (given).
2. From Figure F.5, an annual unit runoff of 50 dam³/km² is determined for 70% probability of exceedence.
3. Convert the annual unit runoff to a volume by multiplying by the effective drainage area of 2.6 km². The resulting volume of 130 dam³ is available for diversion 70% of the time (i.e. in seven out of ten years).

F3E Uses and Limitations

The recommended approach for assessing the water supply potential applies to watersheds that have not been appreciably altered by upstream development. However, the approach can be modified to estimate the uncommitted (i.e. available) water supply potential in a basin that has upstream users. Calculate the "present use" annual runoff by deducting allocated upstream uses (including average annual project evaporation losses) from the natural annual runoffs derived from Figures F.2 to F.8.
Provincial water rights licensing agencies allocate uses and average reservoir evaporation losses for projects.

The relationships that have been developed for this manual are very general and do not reflect conditions that may be experienced in unique basins. Complicated systems or apparently unique situations should be referred to a qualified professional for assessment.

F-4 HYDROLOGY SUMMARY REPORT

The hydrological investigations performed by the project designer are summarized in a standard "Hydrology Summary Report" provided in Figure F.18. The report briefly lists the various parameters used in the hydrology studies and specifies the flood potential and water supply potential.

This report provides the basis for conducting the water supply assessment to select the reservoir Full Supply Level (FSL) and for sizing the spillway system and is considered a necessary input for the project design.

Table F.1 Monthly Distribution of Gross Evaporation and Precipitation in the Canadian Prairies

<table>
<thead>
<tr>
<th>Month</th>
<th>Monthly Distribution as a Percentage of Annual Gross Evaporation</th>
<th>Monthly Distribution as a Percentage of Annual Precipitation</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>0.0</td>
<td>4.5</td>
</tr>
<tr>
<td>February</td>
<td>0.0</td>
<td>4.0</td>
</tr>
<tr>
<td>March</td>
<td>0.0</td>
<td>4.5</td>
</tr>
<tr>
<td>April</td>
<td>6.5</td>
<td>6.0</td>
</tr>
<tr>
<td>May</td>
<td>16.5</td>
<td>10.5</td>
</tr>
<tr>
<td>June</td>
<td>18.5</td>
<td>17.0</td>
</tr>
<tr>
<td>July</td>
<td>20.0</td>
<td>16.0</td>
</tr>
<tr>
<td>August</td>
<td>19.0</td>
<td>13.0</td>
</tr>
<tr>
<td>September</td>
<td>13.0</td>
<td>10.0</td>
</tr>
<tr>
<td>October</td>
<td>6.5</td>
<td>5.5</td>
</tr>
<tr>
<td>November</td>
<td>0.0</td>
<td>4.5</td>
</tr>
<tr>
<td>December</td>
<td>0.0</td>
<td>4.5</td>
</tr>
</tbody>
</table>
\begin{align*}
\log Q_2 &= 0.68879 \log (EDA) - 0.79512 \\
\log Q_5 &= 0.70957 \log (EDA) - 0.33964 \\
\log Q_{10} &= 0.71559 \log (EDA) - 0.12846 \\
\log Q_{20} &= 0.71711 \log (EDA) + 0.03746 \\
\log Q_{50} &= 0.71863 \log (EDA) + 0.21923 \\
\log Q_{100} &= 0.71852 \log (EDA) + 0.33504
\end{align*}

\textbf{Example:} For an effective drainage area of 20 km\(^2\),

\begin{align*}
Q_2 &= 1.3 \text{ m}^3/\text{s}, & Q_5 &= 3.8 \text{ m}^3/\text{s}, & Q_{10} &= 6.3 \text{ m}^3/\text{s}, \\
Q_{20} &= 9.3 \text{ m}^3/\text{s}, & Q_{50} &= 14 \text{ m}^3/\text{s}, & Q_{100} &= 19 \text{ m}^3/\text{s}.
\end{align*}

\textbf{FIGURE F1 FLOOD DETERMINATION EQUATIONS}
FIGURE F9  MAXIMUM NUMBER OF CONSECUTIVE YEARS IN THE 43 - YEAR PERIOD 1950 - 92 THAT THE ANNUAL RUNOFF VOLUME WAS ZERO
FIGURE F12  MAXIMUM NUMBER OF CONSECUTIVE YEARS IN THE 43 - YEAR PERIOD 1950 - 92 THAT THE ANNUAL RUNOFF VOLUME WAS LESS THAN 50% OF THE MEDIAN
FIGURE F13  MAXIMUM NUMBER OF CONSECUTIVE YEARS IN THE 43-YEAR PERIOD 1950-92 THAT THE ANNUAL RUNOFF VOLUME WAS LESS THAN 75% OF THE MEDIAN
FIGURE F14  MAXIMUM NUMBER OF CONSECUTIVE YEARS IN THE 43 - YEAR PERIOD 1950 - 92 THAT THE ANNUAL RUNOFF VOLUME WAS LESS THAN THE MEDIAN
FIGURE F16 MEAN ANNUAL PRECIPITATION (mm) ON THE PRAIRIES FOR THE 30-YEAR PERIOD 1961-90
Figure F17  Mean Annual Ice Thickness for Small Prairie Water Bodies
Flood Potential

<table>
<thead>
<tr>
<th>Peak Flow (m³/s)</th>
<th>( \log Q_2 )</th>
<th>( \log Q_5 )</th>
<th>( \log Q_{10} )</th>
<th>( \log Q_{20} )</th>
<th>( \log Q_{50} )</th>
<th>( \log Q_{100} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1:2</td>
<td>0.68879 log (EDA) - 0.79512</td>
<td>log Q_5 = 0.70957 log (EDA) - 0.33964</td>
<td>log Q_{10} = 0.71559 log (EDA) - 0.12846</td>
<td>log Q_{20} = 0.71711 log (EDA) + 0.03746</td>
<td>log Q_{50} = 0.71863 log (EDA) + 0.21923</td>
<td>log Q_{100} = 0.71852 log (EDA) + 0.33504</td>
</tr>
<tr>
<td>1:5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1:10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1:20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1:50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1:100</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

where: \( Q_2 = 1:2 \) maximum annual instantaneous flood peak (m³/s)
EDA = effective drainage area (km²)

Water Supply Potential

Effective Drainage Area: ______ km²

Median Annual Unit Runoff: ______ dam³/km² (Figure F.2)

MEDIAN ANNUAL RUNOFF: ______ dam³ (EDA * MAUR)

<table>
<thead>
<tr>
<th>Range of Low Annual Runoff Volumes</th>
<th>Maximum Number of Consecutive Years (Figures F.9 to F.14)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zero</td>
<td>Zero</td>
</tr>
<tr>
<td>&lt; 10% of the median</td>
<td>&lt; 10% of the median</td>
</tr>
<tr>
<td>&lt; 25% of the median</td>
<td>&lt; 25% of the median</td>
</tr>
<tr>
<td>&lt; 50% of the median</td>
<td>&lt; 50% of the median</td>
</tr>
<tr>
<td>&lt; 75% of the median</td>
<td>&lt; 75% of the median</td>
</tr>
<tr>
<td>&lt; the median</td>
<td>&lt; the median</td>
</tr>
</tbody>
</table>

Incremental Range of Low Annual Runoff Volumes (% of Median) | Annual Runoff Volume to Project Site (% of Median) | (dam³) | Number of Years Applicable *
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Zero</td>
<td>0</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>0% to 10% of the median</td>
<td>5%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10% to 25% of the median</td>
<td>17.5%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>25% to 50% of the median</td>
<td>37.5%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>50% to 75% of the median</td>
<td>62.5%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>75% to 100% of the median</td>
<td>87.5%</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Determined as the difference between each interval of Consecutive Years

FIGURE F.18 HYDROLOGY SUMMARY REPORT
Critical Low Runoff Sequence:
(Note: First and Last Year of Sequence = Median Annual Runoff)

<table>
<thead>
<tr>
<th>Year 1</th>
<th>Year 2</th>
<th>Year 3</th>
<th>Year 4</th>
<th>Year 5</th>
<th>Year 6</th>
<th>Year 7</th>
<th>Year 8</th>
<th>Year 9</th>
<th>Year 10</th>
<th>Year 11</th>
<th>Year 12</th>
<th>Year 13</th>
<th>Year 14</th>
</tr>
</thead>
</table>

Mean Annual Gross Evaporation: _______ mm (Figure F.15)

Monthly Distribution of Gross Evaporation:

<table>
<thead>
<tr>
<th>Jan</th>
<th>Feb</th>
<th>Mar</th>
<th>Apr</th>
<th>May</th>
<th>June</th>
<th>July</th>
<th>Aug</th>
<th>Sept</th>
<th>Oct</th>
<th>Nov</th>
<th>Dec</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monthly Dist (%)</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>6.5</td>
<td>16.5</td>
<td>18.5</td>
<td>20.0</td>
<td>19.0</td>
<td>13.0</td>
<td>6.5</td>
<td>0</td>
</tr>
</tbody>
</table>

Mean Annual Precipitation: _______ mm (Figure F.16)

Monthly Distribution of Precipitation:

<table>
<thead>
<tr>
<th>Jan</th>
<th>Feb</th>
<th>Mar</th>
<th>Apr</th>
<th>May</th>
<th>June</th>
<th>July</th>
<th>Aug</th>
<th>Sept</th>
<th>Oct</th>
<th>Nov</th>
<th>Dec</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monthly Dist (%)</td>
<td>4.5</td>
<td>4.0</td>
<td>4.5</td>
<td>6.0</td>
<td>10.5</td>
<td>17.0</td>
<td>16.0</td>
<td>13.0</td>
<td>10.0</td>
<td>5.5</td>
<td>4.5</td>
</tr>
</tbody>
</table>

Monthly Distribution of Net Evaporation (Gross Evaporation - Precipitation)

<table>
<thead>
<tr>
<th>Jan</th>
<th>Feb</th>
<th>Mar</th>
<th>Apr</th>
<th>May</th>
<th>June</th>
<th>July</th>
<th>Aug</th>
<th>Sept</th>
<th>Oct</th>
<th>Nov</th>
<th>Dec</th>
</tr>
</thead>
<tbody>
<tr>
<td>mm</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Mean Annual Ice Thickness: _______ metres (Figure F.17)

Reservoir Capacity: _______ dam³  Reservoir Dead Storage Capacity: _______ dam³

Project Annual Demand: _______ dam³

<table>
<thead>
<tr>
<th>Jan</th>
<th>Feb</th>
<th>Mar</th>
<th>Apr</th>
<th>May</th>
<th>June</th>
<th>July</th>
<th>Aug</th>
<th>Sept</th>
<th>Oct</th>
<th>Nov</th>
<th>Dec</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monthly Dist (%)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Result of Project Simulation: _______________________________________

FIGURE F.18 HYDROLOGY SUMMARY REPORT (con’t)
F-5 DEFINITIONS

Annual Unit Runoff

The annual unit runoff at a site is determined as the annual runoff volume at the site for a particular year, divided by the effective drainage area. The units are \(\text{dam}^3/\text{km}^2\).

Correlation Coefficient

A measure of the statistical confidence of a relationship. A value of zero means no relationship, while the closer to 1.0 the better the relationship and the higher the prediction abilities of the relationship.

Critical Low Runoff Sequence

Severe low runoff condition, usually several years, that is deemed to have a reasonable chance of occurrence.

Cunnane Plotting Positions

The predetermined position (determined by an equation) along the x-axis on probability paper for each ranked value (e.g. peak flows) in a frequency analysis.

Dead Storage

Reservoir storage below which no further withdrawal is made, determined by the invert of the riparian, water supply intake, recreation or fishery requirement, etc.

Multiple Regression Analysis

The development of relationships (equations) between one dependent variable (e.g. flood peaks) and more than one independent variable (e.g. effective drainage areas, basin slopes, etc.).

Probability of Exceedence

Chance of being equalled or exceeded. An event with a probability of exceedence of 70% means that in any one year, there is a 70% probability that the magnitude of the event will be equalled or exceeded.

Statistical Confidence

The confidence that can be placed on a relationship or equation based on such statistical parameters as: correlation coefficient, standard error of estimate, etc.
REFERENCES

Bell, B.J., February, 1994, *Annual Unit Runoff on the Canadian Prairies*, Hydrology Report #135, Agriculture and Agri-Food Canada, Prairie Farm Rehabilitation Administration, Engineering and Sustainability Service, Hydrology Division, Regina, Saskatchewan.


Bell, G.W., March, 1994, *Historical Low Annual Runoff for the Canadian Prairies*, Hydrology Report #137, Agriculture and Agri-Food Canada, Prairie Farm Rehabilitation Administration, Engineering and Sustainability Service, Hydrology Division, Regina, Saskatchewan.


Unpublished data, May, 1994, *"Runoff Data for Small Plots at the Agriculture and Agri-Food Canada Swift Current Research Station"*, Agriculture and Agri-Food Canada, Research Branch, Swift Current Research Station, Swift Current, Saskatchewan.


HYDROLOGY SUMMARY REPORT

PROJECT TITLE/OWNER: ________________________
LAND LOCATION: ____________________________
MAP No.: ____________________________
DATE: ____________________________
TECHNICIAN: ____________________________

**Flood Potential**

<table>
<thead>
<tr>
<th>Peak Flow (m³/s)</th>
<th>log Q₂   = 0.68879 log (EDA) - 0.79512</th>
</tr>
</thead>
<tbody>
<tr>
<td>1:2</td>
<td>log Q₅   = 0.70957 log (EDA) - 0.33964</td>
</tr>
<tr>
<td>1:5</td>
<td>log Q₁₀  = 0.71559 log (EDA) - 0.12846</td>
</tr>
<tr>
<td>1:10</td>
<td>log Q₂₀  = 0.71711 log (EDA) + 0.03746</td>
</tr>
<tr>
<td>1:20</td>
<td>log Q₅₀  = 0.71863 log (EDA) + 0.21923</td>
</tr>
<tr>
<td>1:50</td>
<td>log Q₁₀₀ = 0.71852 log (EDA) + 0.33504</td>
</tr>
</tbody>
</table>

where: Q₂ = 1:2 maximum annual instantaneous flood peak (m³/s)
EDA = effective drainage area (km²)

**Water Supply Potential**

Effective Drainage Area: _______ km²

Median Annual Unit Runoff: _______ dam³/km² (Figure F.2)

MEDIAN ANNUAL RUNOFF: _______ dam³  (EDA * MAUR)

<table>
<thead>
<tr>
<th>Range of Low Annual Runoff Volumes</th>
<th>Maximum Number of Consecutive Years (Figures F.9 to F.14)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zero</td>
<td>Zero</td>
</tr>
<tr>
<td>&lt; 10% of the median</td>
<td>&lt; 10% of the median</td>
</tr>
<tr>
<td>&lt; 25% of the median</td>
<td>&lt; 25% of the median</td>
</tr>
<tr>
<td>&lt; 50% of the median</td>
<td>&lt; 50% of the median</td>
</tr>
<tr>
<td>&lt; 75% of the median</td>
<td>&lt; 75% of the median</td>
</tr>
<tr>
<td>&lt; the median</td>
<td>&lt; the median</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Incremental Range of Low Annual Runoff Volumes (% of Median)</th>
<th>Annual Runoff Volume to Project Site</th>
<th>Number of Years Applicable *</th>
</tr>
</thead>
<tbody>
<tr>
<td>(% of Median)</td>
<td>(% of Median)</td>
<td>(dam³)</td>
</tr>
<tr>
<td>Zero</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>0% to 10% of the median</td>
<td>5%</td>
<td>5%</td>
</tr>
<tr>
<td>10% to 25% of the median</td>
<td>17.5%</td>
<td>62.5%</td>
</tr>
<tr>
<td>25% to 50% of the median</td>
<td>37.5%</td>
<td>37.5%</td>
</tr>
<tr>
<td>50% to 75% of the median</td>
<td>62.5%</td>
<td>37.5%</td>
</tr>
<tr>
<td>75% to 100% of the median</td>
<td>87.5%</td>
<td>37.5%</td>
</tr>
</tbody>
</table>

* Determined as the difference between each interval of Consecutive Years
HYDROLOGY SUMMARY REPORT (con’t)

Critical Low Runoff Sequence:
(Note: First and Last Year of Sequence = Median Annual Runoff)

<table>
<thead>
<tr>
<th>Year 1</th>
<th>Year 2</th>
<th>Year 3</th>
<th>Year 4</th>
<th>Year 5</th>
<th>Year 6</th>
<th>Year 7</th>
<th>Year 8</th>
<th>Year 9</th>
<th>Year 10</th>
<th>Year 11</th>
<th>Year 12</th>
<th>Year 13</th>
<th>Year 14</th>
</tr>
</thead>
</table>

Mean Annual Gross Evaporation: _____ mm (Figure F.15)

Monthly Distribution of Gross Evaporation:

<table>
<thead>
<tr>
<th>Jan</th>
<th>Feb</th>
<th>Mar</th>
<th>Apr</th>
<th>May</th>
<th>June</th>
<th>July</th>
<th>Aug</th>
<th>Sept</th>
<th>Oct</th>
<th>Nov</th>
<th>Dec</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>6.5</td>
<td>16.5</td>
<td>18.5</td>
<td>20.0</td>
<td>19.0</td>
<td>13.0</td>
<td>6.5</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Mean Annual Precipitation: _____ mm (Figure F.16)

Monthly Distribution of Precipitation:

<table>
<thead>
<tr>
<th>Jan</th>
<th>Feb</th>
<th>Mar</th>
<th>Apr</th>
<th>May</th>
<th>June</th>
<th>July</th>
<th>Aug</th>
<th>Sept</th>
<th>Oct</th>
<th>Nov</th>
<th>Dec</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.5</td>
<td>4.0</td>
<td>4.5</td>
<td>6.0</td>
<td>10.5</td>
<td>17.0</td>
<td>16.0</td>
<td>13.0</td>
<td>10.0</td>
<td>5.5</td>
<td>4.5</td>
<td>4.5</td>
</tr>
</tbody>
</table>

Monthly Distribution of Net Evaporation (Gross Evaporation - Precipitation)

<table>
<thead>
<tr>
<th>Jan</th>
<th>Feb</th>
<th>Mar</th>
<th>Apr</th>
<th>May</th>
<th>June</th>
<th>July</th>
<th>Aug</th>
<th>Sept</th>
<th>Oct</th>
<th>Nov</th>
<th>Dec</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Mean Annual Ice Thickness: _____ metres (Figure F.17)

Reservoir Capacity: _____ dam³ Reservoir Dead Storage Capacity: _____ dam³

Project Annual Demand: _____ dam³

<table>
<thead>
<tr>
<th>Jan</th>
<th>Feb</th>
<th>Mar</th>
<th>Apr</th>
<th>May</th>
<th>June</th>
<th>July</th>
<th>Aug</th>
<th>Sept</th>
<th>Oct</th>
<th>Nov</th>
<th>Dec</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Result of Project Simulation: ____________________________________________________________
APPENDIX "G" -

Federal Environmental Assessment Process
Appendix G - Federal Environmental Assessment Process

The Canadian Environmental Assessment Act

The Canadian Environmental Assessment Act (CEAA), came into effect January 1995, replacing the Environmental Assessment and Review Process Guidelines Order (EARPGO). The Act sets out responsibilities and procedures for the environmental assessment of "projects", as defined by CEAA, involving the federal government and applies to projects for which the federal government holds decision-making authority either as a proponent, land administrator, funding agency or regulator. When projects receive federal funding, under PFRA's Rural Water Development Program (RWDP) for instance, the environmental assessment process must be applied.

The provision of technical assistance alone can, in some cases, also require the application of CEAA. This is the case if assistance is given on a well defined project and leads to a reduction in cost by the proponent. For example, if PFRA designs a small dam for a landowner and supervises construction, such assistance is considered equivalent to cash and PFRA would be required to screen the project.

Under CEAA, a "project" is defined as either 1) an undertaking in relation to a physical work, such as any proposed construction, operation, modification, or decommissioning (e.g. construction of a small dam) or 2) a physical activity not relating to a physical work that is listed in the CEAA regulations.

Primary objectives of environmental assessment are to:

- ensure that the environmental effects of projects receive consideration before federal agencies take action;
- encourage federal agencies to take actions that promote sustainable development;
- ensure that projects in Canada or on federal lands do not cause adverse effects outside the jurisdictions in which they are carried out; and
- provide an opportunity for public participation in the environmental assessment process.

The federal environmental assessment process is guided by the principles of:

- Early application - the process should be applied early in project planning and before irrevocable decisions (e.g. providing final funding approval) are made;
- Accountability - the self-assessment of projects for environmental effects by federal agencies is a cornerstone of the process;
- Efficiency and cost-effectiveness - federal and provincial agencies should coordinate assessment so that each project undergoes only one environmental assessment and the level of effort should match the scale of the project's likely effects.
- G2 -

- **Open and participatory** - public participation is an important element of a balanced environmental assessment process.

This Appendix outlines the responsibilities of PFRA in fulfilling requirements under *CEAA*.

**Responsibilities**

**PFRA** - The **PFRA Environmental Assessment User’s Guide** outlines general environmental assessment policy and procedures that apply to all PFRA employees and "projects" as defined by *CEAA*. Copies of the guide are available in each District and Regional office, the Land Management Branch, and the technical centres as well as through the PFRA Information Centre in Regina.

The construction, operation and decommissioning of small dams and weirs for which PFRA is either the proponent or provides funding or an interest in land generally will require the completion of a screening report prior to final approval. Routine maintenance of such structures can be excluded even if funded by PFRA however, major modifications may need to be screened.

Electronic forms for screening reports and exclusions have been created in Word Perfect and are available to all PFRA staff (see Figures G.1 and G.2 for examples of the blank forms).

Within PFRA, lead responsibility for conducting the environmental assessment of projects rests with the project manager. For projects funded under RWDP, the project manager is usually a District Head of Water or the District Engineer. Screening reports must be approved by whoever has financial signing authority for the project. Exclusion Decision Statements may be signed by the project manager or by whomever he/she delegates the task to.

The PFRA Regional Environmental Officers (REOs) will be responsible for providing process advice to project managers. The REOs will also liaise with other federal environmental assessment officers and the respective provincial environmental assessment agencies in accordance with the **Federal Coordination Regulations** (Refer to Figure G.3). Liaison will include circulation of the Project Description and additional information as requested. Upon receipt of the signed screening report, the REOs will register the project in AAFC’s electronic data base in accordance with *CEAA* and forward the original document to the PFRA Information Centre for cataloguing.

The attached flowchart (Figure G.4) tracks the steps in the federal Environmental Assessment Process for small dams.

In general, the PFRA project manager will prepare the Project Description in accordance with the **Federal Coordination Regulations** under *CEAA* and will also be responsible for the preparation of the Screening Report or Exclusion Decision
Statement. The project manager may contact federal and provincial regulators (e.g. Environment Canada, Fisheries and Oceans, provincial environment departments) on behalf of the proponent to acquire the necessary approval permits or licenses.

Proponent - As defined by CEAA, proponent means the person, body, federal authority or government that proposes the project. For small dam projects, the owner is generally the proponent and owner of the dam or weir, and is responsible for obtaining environmental approvals from provincial, federal and municipal agencies. In some cases, PFRA may be the owner or proponent. When the proponent is a private landowner, PFRA can advise on what approvals are necessary and can assist in acquiring the necessary approvals. The proponent is also responsible for ensuring that any neighbours who might be affected are well informed about the proposal and provided with opportunities to express concern or support. Contact with the public should be documented, including concerns, and information provided to the PFRA project manager for inclusion in the screening report. The proponent (small dam owner) may be responsible for monitoring of the construction site for several years after construction to ensure that recommended mitigation measures are effective.

Project managers are required to apply CEAA when PFRA is the proponent or owner of a small dam, provides funding or technical service on a well defined project, or grants an interest in land.

Information Requirements

"Environment", as defined by CEAA, includes land, water, air, all organic and inorganic matter and living organisms, and the interacting natural systems that connect these components.

"Environmental effect" means a) any change the project may cause in the environment, including any effect of any change on health and socio-economic conditions, on physical and cultural heritage, on the current use of lands and resources for traditional purposes by aboriginal persons, or on any structure, site or thing of historical, archaeological, paleontological or architectural significance and b) any change to the project that may be caused by the environment.

Generally, an environmental assessment report must answer the questions who, what, where, when, how and why. The attached screening report format (Figure G.5) provides headings and subheadings which may be included in a screening for a small dam project.

1) Who - Who is the project proponent (e.g. individual farmer) and what federal agencies have decision-making responsibility for the proposed project (e.g., PFRA)? Who has lead responsibility for coordinating the process among the federal agencies? Who, apart from the proponent and responsible federal agencies, has a direct interest in the project?
2) **What** - What is the nature of the project being proposed? What alternatives have been considered? A detailed physical description of the proposed project is required, including preliminary design drawings, plans, etc. Preliminary information, provided by PFRA in consultation with the proponent, is to be included in a Project Description prepared by the PFRA project manager for circulation by the PFRA Regional Environmental Officer to other federal and provincial agencies that are likely to have an interest in the project.

3) **Where** - Where will the project be located? Legal land description is to be included as well as relative location to major landscape features (e.g. rivers). Use of topographic maps and air photos is encouraged. Describe where the project is in the environment, making specific reference to water bodies, erodible land, native stands of vegetation, critical wildlife habitat, and other sensitive lands.

4) **When** - When will construction of the project be started/finished, for how long will it be operated, when will it be decommissioned? If the project is to be operated for a short term only, (<20 years) information should be included on how the project will be decommissioned. Scheduling of construction to avoid critical spawning and nesting periods is recommended if appropriate.

5) **How** - How will the project be constructed, operated or decommissioned? What construction methods will be used and what mitigation measures will be used to minimize environmental damage.

6) **Why** - Why is the project being proposed? What needs will the project fill. This information is to be provided to the PFRA project manager by the proponent.

**Level of Effort**

The level of effort for an environmental assessment increases from an "Exclusion" through a "Screening" to a "Comprehensive Study". With rare exception, small dam projects will be screened. Virtually no small dams will require a Comprehensive Study.

**Exclusion** - If the project consists of maintenance of an existing structure, the project may be "excluded" from the process in accordance with the *Exclusion List Regulation*. Excluded projects have been predetermined to have insignificant adverse environmental effects. In such cases a PFRA Environmental Exclusion Decision Statement is completed by the project manager and the appropriate form placed on the project file. The Exclusion Form does not need to be sent to the REO.
Project managers should be aware that other provincial, municipal or federal environmental approvals may still be required even though a project has been excluded under CEAA.

**Screening** - In the large majority of cases, PFRA will be required to "screen" the proposal and make its decision as to the significance of potential environmental impacts before providing funding (refer to the Screening Decision block on Figure G.2). The screening process is carried out in coordination with other federal agencies that may have an interest in the project and with the provincial environmental assessment process. A generic screening report format is attached (Figure G.5). The signed screening report is sent to the REO for registration and forwarding to the PFRA Information Centre and a copy is retained on the project file.
## PFRA ENVIRONMENTAL EXCLUSION DECISION STATEMENT

<table>
<thead>
<tr>
<th>Project Title</th>
<th>PFRA File Number</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>District/Pasture</th>
<th>Legal Land Description</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Financial Authority Document</th>
<th>Federal Trigger(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Funding</td>
</tr>
<tr>
<td></td>
<td>Proponent</td>
</tr>
</tbody>
</table>

### EXCLUSION LIST REFERENCE

[The following projects are a condensed list of Regulation Subsections only. Refer to the Canadian Environmental Assessment Act Exclusion List Regulations (SOR/94-639) for a complete listing of projects for which an environmental assessment is not required.]

#### Part I - General
1. Maintenance/Repair of Existing Physical Work
2. Operation of an Environmentally Approved Existing Physical Work
3. Building Construction
4. Building Expansion/Modification
5. Scientific Data Collection
6. Sidewalk/Parking Lot Construction
7. Sidewalk/Parking Lot Expansion/Modification
8. Fence Expansion/Modification
9. Hook-Up (pipeline) Construction/Expansion
10. Signage Construction/Expansion
11. Road Expansion/Modification
12. Building Demolition

#### Part II - Agriculture
18. Irrigation Structure Modification
19. Domestic/Farm Water Supply
20. Centre Pivot/Side Roll Sprinkler

#### Part V - Water Projects
33. Fish Habitat Structure

#### Part VI - Transportation
42. Culvert Modification

#### Other Projects
- Emergency Response

An environmental assessment is not required for the above proposed project in accordance with the CANADIAN ENVIRONMENTAL ASSESSMENT ACT, Exclusion List Regulations (SOR/94-639).

__________________________

Authorizing Officer

__________________________

Date

Figure G.1
PFRA ENVIRONMENTAL ASSESSMENT
PROJECT SCREENING REPORT

1. PROPOSAL INFORMATION

<table>
<thead>
<tr>
<th>Project Title</th>
<th>PFRA File Number</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>FEAI Reference Number</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Proponent</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Location</th>
<th>PFRA Contact</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Overview</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
</tbody>
</table>

2. SCREENING DECISION

- Project is unlikely to cause significant adverse environmental effects taking into account appropriate mitigation. PFRA may take action to allow project to proceed.
- Project is likely to cause significant adverse environmental effects. PFRA may not take any action to allow project to proceed.
- Uncertain whether project would cause significant adverse environmental effects. Refer to Ministry of the Environment for panel review or mediation.
- Uncertain whether significant adverse environmental effects could be justified under the circumstances. Refer to the Ministry of the Environment for panel review or mediation.
- Public concern merits referral to Ministry of the Environment for panel review or mediation.

______________________________  ____________________________
Authorizing Officer            Date
Figure G.3

*Canadian Environmental Assessment Act - Federal Coordination Regulations*

The Regulations Respecting the Coordination by Federal Authorities of Environmental Assessment Procedures and Requirements (the Federal Coordination Regulations), under the *Canadian Environmental Assessment Act*, came into force as of April 8, 1997. The new regulations are intended to ensure that federal environmental assessment is efficiently coordinated among federal authorities reducing the likelihood of more than one assessment being done for the same project. The Regulations contain the following key elements:

(a) identification and notification of federal authorities that may be involved in a project as responsible authorities or as expert departments according to a timeline;
(b) consultation among federal authorities regarding the scope of the environmental assessment for the project;
(c) release of environmental assessment determinations by all responsible authorities according to an agreed schedule; and
(d) coordination of all responsible authorities’ interests and involvements in comprehensive study recommendations.

This regulation applies to all proposals defined as "projects" under CEAA which cannot be excluded and therefore must be screened or undergo a Comprehensive Study.

The Regional Environmental Officers will be responsible for acting as a liaison between PFRA, other federal departments and the respective provincial environmental assessment branches, when appropriate, in accordance with the regulation and PFRA Policy. This will be done in consultation with staff in the Regional and District Offices and Technical Centres.

**Project managers will be responsible for providing a project description to the REO, as early as possible in the planning process,** which can be forwarded by fax to other federal agencies, the provincial environmental assessment branch and the Agency. The definition of "project description" in the Federal Coordination Regulations is as follows:

"Any information in relation to a project that includes
a) a summary description of the project;
b) information indicating the location of the project and the areas potentially affected by the project;
c) a summary description of the physical and biological environments within the areas potentially affected by the project; and
d) the mailing address and phone number of a PFRA contact person who can provide additional information about the project."
The use of maps and air photos is encouraged. It is not necessary to have
the final design available before requesting whether other federal agencies have an
interest in the proposed project. Should design changes be made late in project
planning, it is unlikely that they will affect whether or not an agency has an
interest in the proposal under CEAA. The REO will, however, advise those
agencies who have expressed an interest in the project of such changes and any
other pertinent information. When more than PFRA has an interest in a project, the
REO, together with the PFRA project manager and other Responsible Authority, will
agree upon the respective roles of the agencies in carrying out the environmental
assessment, the scope of the project and the scope of the assessment.

The timelines for notification are spelled out in the regulation. When a
project is subject to assessment by another jurisdiction, (e.g. the province), a
shorter time frame for consideration is provided. Briefly, if there is NO indication
that a project is subject to assessment by another jurisdiction, a federal authority
legally has 30 working days to determine if it is likely to have a trigger under
CEAA, can provide expert advice, or requires further information. If the project is
subject to assessment by another jurisdiction, the time allowed to make a
determination is reduced to 10 working days. If a federal agency requests further
information, it has a further 10 days to make a determination after receiving the
requested information. What these timelines mean for PFRA project managers, is
that it is very important to begin the environmental assessment process early on in
project planning.

Project managers will, as in the past, still be responsible for contacting
federal and provincial agencies for regulatory approval, either for PFRA or on behalf
of clients. In some instances, the contact person may be the same as for
environmental approval. Project managers will also still be responsible for
preparing the screening report which is to be signed by the person having financial
signing authority and forwarding it to the REO for registration. It is preferable that
the project description be distributed prior to contact with federal regulators.
PFRA Environmental Assessment Process for Small Dams

CEAA PROCESS FLOWCHART
Fig G.4

ALL PFRA PROPOSALS

Does PFRA hold decision making authority (i.e., proponent, money, land, law)

- No → CEAA does not apply (no CEAA documentation required)
- Yes

Is proposal a "project" according to CEAA

- No → CEAA does not apply (check appropriate box on Request for Authority Form or other financial form)
- Yes

Is the project listed in the Exclusion List Regulations

- Yes → Project can be excluded from assessment (Complete PFRA Environmental Exclusion Decision Statement and send to project file. Check appropriate box on Request for Authority Form or other financial form)
- No

(Complete PFRA Environmental Assessment Project Screening Report in accordance with PFRA Procedures for Environmental Assessment or Supplementary Procedures; send original to REO for registration and forwarding to the PFRA Information Centre and send copies to project files; check appropriate box on Request for Authority Form or other financial form.)

Decision

- Refer project to Federal Minister of Environment
- Do not proceed with project
- Proceed with project and recommended mitigation/monitoring
A. **BACKGROUND**

The first paragraph includes most of the information needed to complete the public registry, including name of proponent, PFRA CEAA trigger, funding program, type of physical work and related activity, location, "city" as shown on CEAA Ecoregion map, name of water body, month and year construction to start and finish, and any other federal authorities with an interest in the project.

**Background - Example Wording**

Xxx of Someplace, Saskatchewan has applied for federal funding under PFRA’s Rural Water Development Program (RWDP) to develop and construct a small dam to meet irrigation and livestock needs. The project is located at 1/4 S - Twp - R - W4M on the xxx creek, a tributary of the xxx river. The project is located in the xxx ecoregion represented by xxx. Construction of the project is scheduled to begin month/ year and to be completed by month/ year.

As the federal funding agency, PFRA will act as the lead Responsible Authority (RA) in accordance with CEAA and will coordinate the environmental assessment with other federal and provincial agencies as appropriate. No other federal RAs have been identified although Fisheries and Oceans and Environment Canada have offered specialist advice.

PFRA has defined the scope of the project to include the construction and operation of the dam and reservoir.

B. **PROJECT DESCRIPTION**

This section provides a more detailed description of the physical work including size, construction methods, need for or purpose, contribution to sustainable agriculture, etc. Attach more detailed location maps, photos, site plans, and preliminary design drawings.

**Project Description - Subheadings**

- Need for project
- Project owner and operator
- Location and water source
C. DESCRIPTION OF THE ENVIRONMENT

This section could include a description of the topography, soils, natural vegetation, water features, climate and weather, wildlife/fish, land use and cultural features both at the project site and in the surrounding area which could be affected directly by the project. Special attention should be paid to rare or endangered species, and sensitive areas such as wetlands, river valleys, native range, sand dune areas, and known heritage sites.

Information in this section may be obtained from air photos, topographic and soils maps, data bases and maps maintained by provincial and federal environment agencies, or personal communication with provincial/federal environment officials. Site specific data collection by the proponent or PFRA will likely provide additional information.

Description of the Environment - Subheadings

- Water source
  - surface/ground water hydrology
  - supply and quality
- Topography and soils
- Land use and cover
  - cultivated agriculture
  - native pasture/tame forages
Figure G.5 (Continued)

- Fish and fish habitat
- Sensitive areas in reservoir area
  - valley lands
  - erodible lands (steep slopes, dunes)
  - native grasslands
  - wildlife habitat
- Wildlife and wildlife habitat
  - rare and endangered species
- Heritage resources

D. POTENTIAL ENVIRONMENTAL IMPACTS AND MITIGATION

Consider short and long term potential effects of physical activities (construction, operation - including risk of dam failure, decommissioning) on land, water, air, all living organisms (include humans) and cultural features. Indicate if effects will be local or regional in nature. State types of mitigation, including scheduling of construction, avoidance of sensitive areas/features, construction practices such as erosion control measures and handling of hazardous materials, especially around water. Make reference to any recommendations contained in federal or provincial environmental permits.

Conclude with significance decision (e.g. Provided that the proponent meets requirements of name of environmental permit(s) and carries out recommended mitigation, PFRA has determined that the proposed project is unlikely to cause significant adverse environmental effects).

Environmental Impacts and Mitigation - Subheadings

- Potential impact on fish and wildlife
  - mitigation - scheduling, avoidance of sensitive sites, erosion control, recommendations in the fisheries permits, adherence with provincial and federal fish habitat protection guidelines for water control structures, fish passage facility
  - inspection and follow-up monitoring
  - responsibility of contractor contained in tender documents
- Potential impact on heritage resources
  - mitigation - avoidance
- Potential impact on water quantity/quality
  - surface water - potential impact of construction and operation on quantity and quality - construction practices and erosion control measures used to avoid/minimize risk of pollution
  - downstream flows - changes in peak and average flows and quality (e.g., temperature, sediment, DO, etc.)
Figure G.5 (Continued)

- Potential impact on land use
  - wildlife habitat
  - agricultural land (cultivated, pasture/forages)
  - disruption of linear features (e.g. roads, utilities)
  - recreation

E. **PUBLIC INVOLVEMENT**

Document efforts of the proponent to inform and involve the public. Include meetings with neighbours, the RM or town council, media coverage, open houses, etc.

State that PFRA, in accordance with CEAA, will register the project in the public registry.

For information contact PFRA project leader name, address, phone and fax.
REFERENCES
REFERENCES


Durrant, E.F., and S.R. Blackwell, April, 1961, The Magnitude and Frequency of Floods in Alberta, Saskatchewan and Manitoba, Canada Department of Agriculture, Prairie Farm Rehabilitation Administration, Hydrology Division, Regina, Saskatchewan.


PFRA Field Manual, PFRA, Water Development Branch.

Smith C.D., 1985, Hydraulic Structures


Resistance of Soils to Water Erosion

1. Granular Soil

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Flow Velocity (m/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>silt and very fine sand</td>
<td>0.15 - 0.25</td>
</tr>
<tr>
<td>fine to coarse sand</td>
<td>0.3 - 0.6</td>
</tr>
<tr>
<td>fine to coarse gravel</td>
<td>0.75 - 1.2</td>
</tr>
<tr>
<td>cobble</td>
<td>1.5</td>
</tr>
</tbody>
</table>

2. Clay and Clayey Soils

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Flow Velocity (m/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>silty to sand loam</td>
<td>0.55 - 0.6</td>
</tr>
<tr>
<td>sandy loam to loam</td>
<td>0.6 - 0.75</td>
</tr>
<tr>
<td>loam to clay loam</td>
<td>0.75 - 1.1</td>
</tr>
<tr>
<td>stiff clay</td>
<td>1.1 - 1.85</td>
</tr>
<tr>
<td>gravelly clay; graded clay to cobble</td>
<td>1.2 - 2.1</td>
</tr>
</tbody>
</table>

References:


<table>
<thead>
<tr>
<th>HAZARD POTENTIAL ASSESSMENT SUMMARY</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Project Title / Owner:</strong></td>
</tr>
<tr>
<td><strong>Designer:</strong></td>
</tr>
<tr>
<td><strong>Downstream Schematic Map</strong></td>
</tr>
<tr>
<td><strong>Estimated Embankment Height:</strong></td>
</tr>
<tr>
<td><strong>Estimated Storage at Top of Dam:</strong></td>
</tr>
<tr>
<td><strong>Reference Map:</strong></td>
</tr>
<tr>
<td><strong>Office Studies:</strong></td>
</tr>
<tr>
<td><strong>Loss of Life:</strong></td>
</tr>
<tr>
<td><strong>Estimated Flood Damage (1980 $):</strong></td>
</tr>
<tr>
<td><strong>Estimated Other Damage (1980 $):</strong></td>
</tr>
<tr>
<td><strong>Discussion with Project Proponent:</strong></td>
</tr>
<tr>
<td><strong>Hazard Potential Rating:</strong></td>
</tr>
</tbody>
</table>

- Estimated Embankment Height: ________ m
- Estimated Storage at Top of Dam: ________ dam$^3$
- Reference Map: □ yes □ no
- Office Studies: □ yes □ no
- Loss of Life: □ yes □ no
- Estimated Flood Damage (1980 $): __________
- Estimated Other Damage (1980 $): __________
- Discussion with Project Proponent: □ yes □ no
- Hazard Potential Rating: C □ greater than C □
GEOLOGICAL DAMSITE DESCRIPTION - SUMMARY SHEET

<table>
<thead>
<tr>
<th>Project Title / Owner:</th>
<th>Location:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Designer:</th>
<th>Data:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Site or Centreline Designations:

Site comments based on interpretation of the following information:

Field Exploration Data ☐ Soil Maps and Reports ☐ Subsurface Investigation ☐
Local Water Well Logs ☐ Geological Maps and Reports ☐ Air Photo Interpretation ☐

A. Abutments

<table>
<thead>
<tr>
<th>Geological Unit(s) forming abutments: Pre-valley P or Post valley V</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay ☐ Silt ☐ Sand ☐ Gravel ☐ Till ☐ Shale ☐ Sandstone ☐</td>
</tr>
<tr>
<td>Other units present:</td>
</tr>
<tr>
<td>Evidence of abutment instability: None ☐ Dormant ☐ Active ☐</td>
</tr>
<tr>
<td>Evidence of natural seepage: None ☐ Present ☐</td>
</tr>
<tr>
<td>Estimated valley slopes: _____:1</td>
</tr>
</tbody>
</table>

B. Foundation

<table>
<thead>
<tr>
<th>Geological Unit(s) immediately underlying valley floor: Pre-valley P or Post valley V</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay ☐ Gravel ☐ Shale ☐ Silt ☐ Till ☐ Sand ☐ Sandstone ☐</td>
</tr>
<tr>
<td>Other units present:</td>
</tr>
<tr>
<td>Estimated maximum thickness: _____ m</td>
</tr>
<tr>
<td>Evidence of high water table: None ☐ Present ☐</td>
</tr>
<tr>
<td>Evidence of salt concentration on surface: None ☐ Present ☐</td>
</tr>
</tbody>
</table>

C. Reservoir

<table>
<thead>
<tr>
<th>Cultural features affected by reservoir</th>
</tr>
</thead>
<tbody>
<tr>
<td>Buildings: None ☐ Present ☐ Roads: None ☐ Present ☐</td>
</tr>
<tr>
<td>Evidence of valley side instability: None ☐ Dormant ☐ Active ☐</td>
</tr>
</tbody>
</table>

D. Materials of Construction

<table>
<thead>
<tr>
<th>Material</th>
<th>Local (within 200 m)</th>
<th>Within 1 km</th>
<th>Unknown or &gt;1 km</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay or Till</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
</tr>
<tr>
<td>Sand and Gravel</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
</tr>
<tr>
<td>Cobble and Boulders</td>
<td>☐</td>
<td>☐</td>
<td>☐</td>
</tr>
</tbody>
</table>

E. Schematic Valley Cross-Section at Embankment Centreline (attach additional sheets for reservoir cross-sections)
**HYDROLOGY SUMMARY REPORT**

<table>
<thead>
<tr>
<th>Project Title / Owner:</th>
<th>Location:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Designer:</th>
<th>Date:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### General Site Characteristics

<table>
<thead>
<tr>
<th>Effective Drainage Area:</th>
<th>Gross Drainage Area:</th>
</tr>
</thead>
<tbody>
<tr>
<td>mm²</td>
<td>mm²</td>
</tr>
</tbody>
</table>

Watershed Description

Schematic Watershed Map

### Water Supply Potential

Runoff-Draft Region: ________ Median Annual Unit Runoff: ________ dam³/km²

Median Annual Runoff: ________ dam³ Project Demand:

- a) Live Stream Diversion (No Reservoir) -
  - Divertable Annual Volume: 50% of the time ________ dam³; 70% of the time ________ dam³

- b) Storage Condition -
  - Reservoir capacity as a % of Median Annual Runoff: ________; Reservoir Capacity: ________ dam³

  Firm Annual Draft (0% shortages): (Jan. - Dec. Demand) ________ dam³; (May - Aug. Demand) ________ dam³.

  Annual Draft with shortages in 30% of the years: (Jan. - Dec. Demand) ________ dam³; (May - Aug. Demand) ________ dam³.

### Flood Potential

<table>
<thead>
<tr>
<th>Overall Drainability Factor:</th>
<th>Index Flood (1 : 2):</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>________ m³/s</td>
</tr>
</tbody>
</table>

Index Flood Multiplier (1 : 10) ________; (1 : 50) ________; (1 : 100) ________

Rainfall Adjustment Factor (1 : 10) ________; (1 : 50) ________; (1 : 100) ________

Soil Moisture Adjustment Factor (1 : 10) ________; (1 : 50) ________; (1 : 100) ________

Instantaneous Flood Peaks (m³/s) (1 : 10) ________; (1 : 50) ________; (1 : 100) ________
GEOTECHNICAL ASSESSMENT REPORT

Project Title / Owner: Location:
Designer: Date:

Schematic foundation profile and site plan with soil logs and borrow area location on reverse side.

Foundation Conditions

<table>
<thead>
<tr>
<th>Location</th>
<th>Soil Types</th>
<th>Depth of Groundwater (m)</th>
<th>Potential Problems</th>
<th>Suitability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Valley Bottom</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Left Abutment</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Right Abutment</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Upstream</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Downstream</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Final Assessment and Comments:


Borrow Materials

<table>
<thead>
<tr>
<th>Location</th>
<th>Soil Types</th>
<th>Water Content</th>
<th>Optimum Water Content</th>
<th>Suitability</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Final Assessment and Comments:


# PFRA TESTHOLE LOG

<table>
<thead>
<tr>
<th>Depths - m (Underline sample depth)</th>
<th>Soil Classification</th>
<th>Soil Description, Characteristics, Abnormal Conditions, Sample Type, Water Data</th>
<th>Sample Data</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**NOTE** - All samples should be properly labelled with project, site, hole no., sample depth, sample classification and date.
**GRAIN SIZE DISTRIBUTION**

PROJECT ____________________________
LOCATION SAMPLED _____________________
DATE SAMPLED __________________________
SAMPLE NO _____________________________
SAMPLED BY ____________________________

D10 = __________ D30 = __________ D60 = __________
Cu = __________ D15 = __________ D85 = __________
Cc = __________

<table>
<thead>
<tr>
<th>BOULDERS-COBBLIES</th>
<th>GRAVEL</th>
<th>SAND</th>
<th>FINES</th>
</tr>
</thead>
<tbody>
<tr>
<td>coarse</td>
<td>fine</td>
<td>coarse</td>
<td>medium</td>
</tr>
</tbody>
</table>

**CLASSIFICATION**

__________________________
Max. Size mm
**PLAN**

**SECTION B-B**

**SECTION C-C**

**SECTION D-D**

**REINFORCEMENT TABLE**

<table>
<thead>
<tr>
<th>No of</th>
<th>Dia.</th>
<th>6 in.</th>
<th>Total Length</th>
<th>Nominal</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**NOTES**

2. Reinforcement shall be adhered to ends of bars. The ends of bars 50 mm (2") long, shall be perpendicular to the bar.
3. All bars and centroidal bar sizes shall be 15.0 x 200 mm (0.6 x 8") or larger.
4. Clear concrete cover to reinforcement shall be 15 mm unless otherwise specified.
5. All bars are 30 bar diameters.
6. Our lengths are cut to suit in the field.
7. Height of structure is 11/2.