Civil Works for Hydroelectric Facilities

Guidelines for Life Extension and Upgrade

ASCE Hydropower Task Committee

ASCE

CIVIL WORKS FOR HYDROELECTRIC FACILITIES

Guidelines for Life Extension and Upgrade

PREPARED BY
Hydropower Task Committee of the Energy Division of
the American Society of Civil Engineers

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Foreword

Background

Civil Works for Hydroelectric Facilities: Guidelines for Life Extension and Upgrade is an addition to the series of publications by members of the Hydropower Committee of the American Society of Civil Engineers (ASCE) Energy Division.

The Hydropower Committee of ASCE's Energy Division was formed to develop and distribute information on all aspects of hydroelectric power to the industry. The Hydropower Committee concentrates on preparing and publishing "Manuals of Practice," "Guidelines," and "Technical Reports" delineating engineering and scientific issues related to hydroelectric facilities. The Hydropower Committee sees challenges going beyond civil engineering, to include scientists, economists, and other technologists with expertise in peripheral areas.

This task committee was conceived by committee members who volunteered various topics at a meeting of the Hydropower Committee in August 2000.

Applicability

The task committee purpose, as embodied in the mission statement for the committee, is:

"Life Extension and Upgrade of Civil Works for Hydroelectric Facilities" provides guidelines on methodologies and techniques for applying rehabilitation engineering to aging hydroelectric infrastructure. Life extension and upgrade options will be outlined for the various civil features of aging hydroelectric infrastructure by assembling the collective knowledge of the committee and providing examples and engineering references.

This document is educational, providing some guidance on the processes used for life extension and upgrading of civil works. The guidance provided is not intended to be prescriptive, or a codification, baseline, or standard, and should be used with care and prudent judgment.

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The following individuals reviewed the final draft of the document and provided valuable comments and suggestions. The committee appreciates the significant effort of the peer reviewers, which was instrumental in the completion of this document.

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Abbreviations

a.k.a.	Also Known As
AASHTO	American Association of State Highway and Transportation Officials
ADA	Americans with Disabilities Act
AIA	American Institute of Architects
ANSI	American National Standards Institute
ASCE	American Society of Civil Engineers
ASDSO	Association of State Dam Safety Officials
ASME	American Society of Mechanical Engineers
ASTM	American Society of Testing Materials
BLM	Bureau of Land Management
cfm	cubic feet per minute
CFR	Code of Federal Regulations
cfs	cubic feet per second
cy	cubic yard
CZMA	Coastal Zone Management Act
DIN	Deutsches Institut für Normung
EA	Environmental Assessment
EAP	Emergency Action Plan
EIS	Environmental Impact Statement
EPA	Environmental Protection Agency
EPRI	Electric Power Research Institute
ESA	Endangered Species Act
FEMA	Federal Emergency Management Agency
FERC	Federal Energy Regulatory Commission
fps	feet per second
HEC	Hydrologic Engineering Center
HP	Horsepower
ICODS	Interagency Committee On Dam Safety
IDF	Inflow Design Flood
kV	Kilovolt
kW	Kilowatt
kWh	Kilowatt hour
MW	Megawatt
n.d.	No date
NEPA	National Environmental Policy Act
NERC	North American Electric Reliability Council
NGO	Non-Governmental Organization
NHPA	National Historic Preservation Act
NMFS	National Marine Fisheries Service
NPDES	National Pollutant Discharge Elimination System

NWS	National Weather Service
O&M	Operation & Maintenance
PFMA	Probable Failure Mode Analysis
PMF	Probable Maximum Flood
psi	pounds per square inch
RCC	Roller Compacted Concrete
ROW	Right of Way
SCS	Soil Conservation Service
SHPO	State Historic Preservation Office
T&D	Transmission & Distribution
TES	Threatened and Endangered Species
TVA	Tennessee Valley Authority
USACE	United States Army Corps of Engineers
USBR	United States Bureau of Reclamation
USDOE	United States Department of Energy
USFWS	United States Fish & Wildlife Service
USGS	United States Geological Survey
USMFS	United States Marine Fisheries Service
USSD	United States Society on Dams

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

1.0 CHAPTER 1 – INTRODUCTION

1.1 Objective

This book is about extending the service life and improving the performance of hydroelectric civil works. Civil works are the principal project features that form the backbone and structure supporting every hydroelectric project. The civil works discussed in this guideline are divided into four categories of features, as shown on Table 1.1.

Table 1.1 Guideline Subjects

STRUCTURES CHAPTER 4	WATER CONVEYANCES CHAPTER 5	WATER CONTROL DEVICES CHAPTER 6	ANCILLARY SYSTEMS, SAFETY AND SECURITY CHAPTER 7
Intakes Dams Spillways Reservoirs Powerhouses Fish Passage Trash Racks Trash Rakes	Intake Canals Flumes Forebays Tunnels & Shafts Penstocks Tailraces	Gates & Gate Hoists Valves & Operators Flashboards	Civil Work Systems Critical Systems Recreational Safety Project Security

Although focused on hydroelectric projects, this book contains information applicable to other projects without a generation component. Limited to civil works only, this book does not include mechanical or electrical works associated with the power generating equipment, such as turbines, generators, switchgear, and power generation control systems. There are many excellent sources for information on the life-extension, upgrade and modernization of the electrical/mechanical aspects, and the reader is directed to Appendix A for references on hydroelectric engineering available on these components, particularly from the USACE, Bureau of Reclamation, EPRI and HCI.

Extending service life involves any activity that prolongs the life of a particular civil work beyond that expected with routine maintenance. When routine maintenance no longer preserves functionality and serviceability, or becomes too expensive, then extending the service life is an alternative.

Improving performance involves any activity that improves safety, operation, or energy production beyond present performance. This involves enhancing, upgrading or modernizing the function for which a particular feature is designed, e.g. increasing the hydraulic capacity of a penstock or spillway.

The costs of extending service life, or improving performance, are met either by major maintenance funds, or by capital expenditure. While routine maintenance can indeed extend service life and possibly improve performance, it is not the subject of this book. Instead, this book is about extraordinary activities or investments to extend life and improve performance. Such activities are likely to require outages that take civil works temporarily out of service, and thus must be scheduled and planned to minimize generation loss.

As a project ages, life cycle costs often increase to the point that alternatives to maintaining the *status quo* become attractive. Costs associated with routine maintenance and upkeep begin to accelerate as the civil feature ages, especially if decisive action is not taken to arrest deterioration and accelerating costs. The generalized relationships between life cycle costs, relative to their influencing factors, are shown in Figure 1.1.

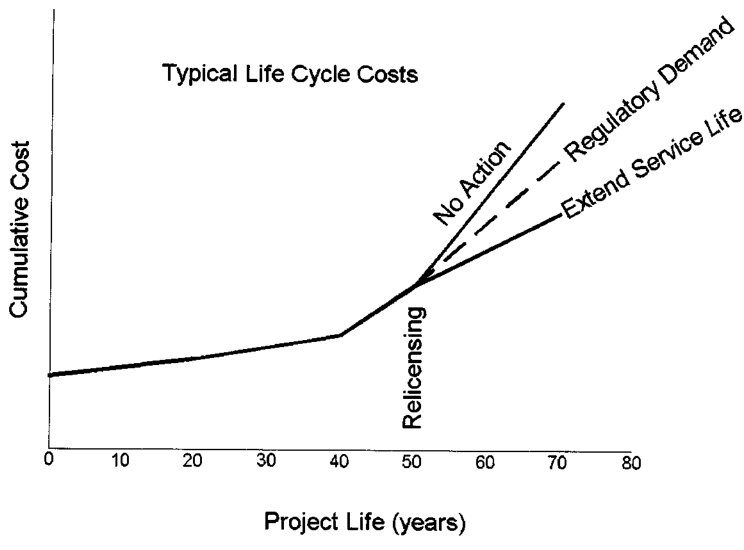


Figure 1.1 Typical Life Cycle Costs (courtesy of Committee Collective Knowledge)

Whereas project life may be well understood as a function of the duration of a license, or of a payback period, service life lacks a widely accepted definition and varies by project feature. A civil work's service life is independent of the life of the project it serves. Many civil works, especially dams, endure long beyond the originally intended project life. Other civil works, such as conveyances, may have much shorter service lives. Floods and earthquakes, as well as disasters like landslides or human error, often shorten the life of a civil work structure.

The distinction between extending service life, and improving performance, often is unclear. Activities described in this book achieve either or both purposes. Remember, "life extension" simply means extending service life; whereas, "upgrade" means improving performance.

1.2 About This Guideline

This book is divided into seven chapters. This chapter introduces the guidelines and summarizes their intent and scope. Planning and Evaluation, Chapter 2, describes the process of extending service life and improving performance, determining project information that contributes to the basis for decision-making, and putting it all together into a condition assessment. Innovative Technologies, Chapter 3, outlines ways to extend the life, and modernize civil components, by the use of innovative technology. Chapter 4 - Structures, Chapter 5 – Water Conveyances, and Chapter 6 – Water Control Devices are the essence of this book. Organized by principal project feature, these chapters describe common problems, their causes, some typical solutions, and over 80 real-world case histories, at various levels of detail of what has worked and what hasn't worked. The reader is informed about problems that have already been solved, with the objective of learning from the experience of others, and thereby eliminating some of the trial and error associated with the steep portion of the "learning curve". Ancillary Systems, Safety and Security, Chapter 7, discusses some other important project features that should be considered during a life extension or upgrade. This includes topics such as instrumentation, flood proofing, oil containment, and project security.

Appendix A is a compilation of useful hydroelectric and dam engineering resources, including specific references from ASCE, EPRI, FERC, HCI Publications, USACE and USBR.

Appendix B describes the various types and sources of information to be used during the planning, evaluation and implementation phases for life extension and upgrade projects.

Appendix C contains several tables on gates, valves, flashboards and flood proofing measures at dams and powerhouses.

Appendix D is a high level summary of Case Histories used in Chapters 4, 5 and 6 by project feature, covering:

- Issues or problems.
- Opportunities for solutions on life extension and upgrade.
- Case history listing by Title and Page Number.

References and resources are contained in three separate locations of the book. A list of technical references, organized by principal project feature, is included at the end of each chapter. References representing some of the "Collective Knowledge" of the committee for each of the features are summarized at the end of each feature. Notations in the text refer to the technical references at the end of each chapter. General hydroelectric resources are contained in Appendix A.

The knowledge base from which this book is derived is a long and rich history of experience in making things last longer, and work better. In a previous document prepared by an ASCE Task committee (ASCE, 1992), the focus was on describing the methods for assessing the condition of the civil works of hydroelectric plants, and on outlining some of the procedures available for civil works rehabilitation and repair.

This current guideline is intended to go further and provide project owners, operators, constructors, and practicing professionals, with techniques and applications that have been successful in extending service life, and improving performance, of hydroelectric civil works. The intended audiences are the practicing engineers who wish to build on the experiences of others, owners and operators, regulatory agencies, and non-governmental agencies.

1.3 Historical Perspective

The engineering of dams and the development of hydropower have a long and colorful history. Although civil works may endure for a long time, many of them have approached the end of their service lives. In the current era, due to social and regulatory concerns, the engineering of new dams in North America has all but ceased; thus rehabilitative engineering is at the forefront.

Civil engineering practice related to dams and hydropower has also advanced. Modern engineering design practices have changed dramatically, and the civil structures of the past are now required to meet 21st century criteria.

These two forces, aging infrastructure and evolving design criteria, have influenced the progress of hydroelectric engineering in the past 50 years. Table 1.2 summarizes some of the important dam and hydropower milestones.

Table 1.2 Some Historical Dam and Hydropower Milestones

YEAR	MILESTONE
2950 to 2750 BC	Sadd el-Kafara Dam (Egypt)- earliest known dam – 37-ft high and over 300 ft long
1824	USACE began water project development on the Ohio and Mississippi Rivers for navigation and flood control
1870	Riveted steel penstocks used
1880-1890	Michigan, Washington, Wisconsin, Oregon, Niagara Falls NY – Early hydroelectric developments
1889	San Mateo Dam (California) completed – First dam built entirely of concrete
1889	South Fork earthen dam failure Johnstown PA (2,209 fatalities)

YEAR	MILESTONE
1901-1902	Federal Water Power Act and establishment of the US Bureau of Reclamation (USBR)
1908	Gravity Dam Design codified
1920	Federal Power Commission (precursor to FERC) and California State Dam Safety established
1933	TVA established – Federal development of water and power for economic development Construction innovation – mass concrete pours for construction of Hoover Dam
1934	Hyrum Dam (Utah) – USBR installs first known piezometers
1935	Federal Power Commission (FPC) authority extended regulating non-federal hydropower development
1949	ASTM A36 Steel introduced
1958	Use of Post-tensioning anchors for rehabilitation
1963	A325 Bolts introduced
1965	FPC order 35 – outlines responsibilities for Dam owners to ensure safe construction and operation
1972	National Dam Inspection Act (Public Law 92-367) passed Required US Army Corps of Engineers to carry out a program of inspection and inventory of most dams in the US
1974	Roller-compacted Concrete (RCC) first used for Dam Construction
1976	Teton Dam (Idaho) failure – 11 fatalities
1977	Toccoa Falls (Kelly Barnes) Dam failure (Georgia) – 39 fatalities Federal Power Commission reorganized as Federal Energy Regulatory Commission
1977-1981	Safety inspections – Over 8,000 dams under the ASCE program were inspected and roughly one third of them were found to be unsafe
1977	USDOE Small Hydroelectric Demonstration program – Spurred development of hydroelectric at existing dams by providing low cost loans
1978	Enactment of the Public Utility Regulatory Policy Act of 1978 – Provided tax credits for development of small hydroelectric before 1989 and incentive rates for Non-utility generators (NUGs)
1979	Executive Order 12148 signed – FEMA assigned the responsibility for coordinating and promoting dam safety
1981	Regulations Governing Safety of Water Power Projects and Project Works, FERC Order No. 122 – FERC issues regulations requiring independent consultant inspections and spillway design flood evaluations
1983	National Hydropower Association formed to promote hydropower
1985	Association of State Dam Safety Officials (ASDSO), and Interagency Committee on Dam Safety (ICODS), and National Safety Council formed to address flood and earthquake concerns
1986	Enactment of Electric Consumers Protection Act (ECPA) – Gave broad authority to environmental agencies and NGO's in FERC relicensing

YEAR	MILESTONE
1992	Class of 1992 – nearly 100 FERC hydro projects up for license renewal – First wave of relicensing, resulting from 50 year licenses expiring from the 1940s
1996	National Performance of Dams Program (NPDP) started and National Dam Safety Program established : Section 215 of Public Law 104-303
1996	California and Pennsylvania first to enact electric competition for consumers and restructuring of electric utility industry
2000	National Energy Policy – First rethink of energy policy initiatives in 20 years – promotes renewables
2001	September 11 – FERC and dam safety authorities re-think pubic and project safety
2002	FERC Initiative – Probable Failure Mode Analysis (PFMA)
2002	US Bureau of Reclamation – 100th Anniversary American Society of Civil Engineers – 150th Anniversary
2003	Changes to FERC relicensing process to expedite process

(Committee Collective Knowledge)

1.3.1 Dam and Hydropower Development & Aging Infrastructure

The earliest known dam, Sadd el-Kafara, was constructed in Egypt. About 37 ft high and over 300 ft long, it is believed to have served from 2950-2750 BC – indeed a remarkable service life.

Hydroelectric development in the United States began in 1880 when Michigan's Grand Rapids Electric Light and Power Company generated electricity by a dynamo. Niagara Falls followed in 1881 by lighting city street lamps powered by hydropower. By 1889, 200 electric plants in the U.S. used waterpower for some, or all, of their generation.

In 1901, the first Federal Water Power Act was enacted, and in 1902 the Bureau of Reclamation was established. By 1907, 15% of all electric generating capacity in U.S. was provided by hydropower.

The Federal Power Act established the Federal Power Commission (now the Federal Energy Regulatory Commission [FERC]). The Federal Power Commission authority was extended to all hydroelectric projects built by utilities engaged in interstate commerce in 1935.

Since then, several other government agencies such as the Tennessee Valley Authority, the Bonneville Power Administration, the Corps of Engineers, and the Bureau of Reclamation have been formed, and continue to operate, and maintain, many hydroelectric projects.

The "Golden Age" of dam building in the United States (US) occupied nearly three decades. The number of dams constructed in the US, as reported by the US Army

Corps of Engineers' National Dam Inventory in 1996, was just over 75,000. As seen in Figure 1.2, the majority of these (41,000) were constructed in the decades of the 1950's, 60's, and 70's. Hydroelectric facilities exist at only a fraction of the total – at approximately 2,300 dams.

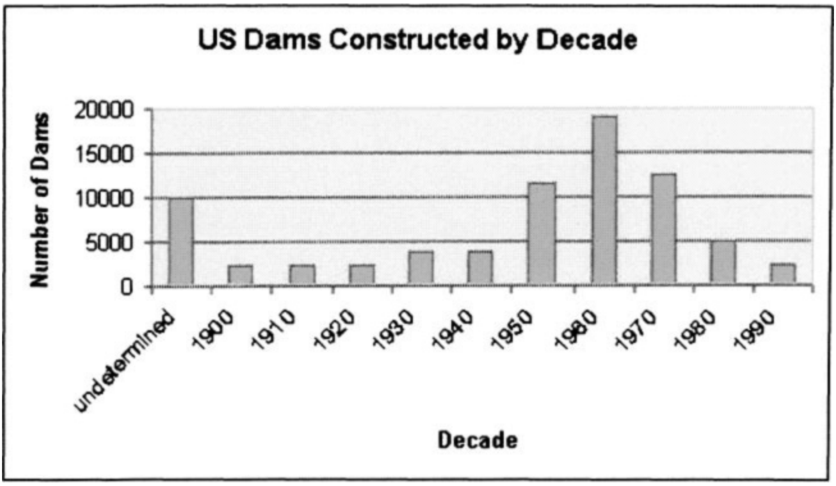


Figure 1.2 US Dams Constructed by Decade (courtesy of Portland District, USACE)

Private hydropower development has always contributed significantly to overall US energy capacity. The industry saw a slight resurgence in the late 1980's as a result of the energy crisis. Today, there is an emerging interest in redevelopment, and the potential of new hydropower at existing dam sites. Currently, about 7% of US electric capacity comes from hydropower, in the form of approximately 80,000 MW of conventional capacity, and 18,000 MW of pumped storage.

Many dams and hydropower projects are reaching an age when rehabilitation, as well as relicensing for FERC regulated projects, is required. Many dams constructed in the 1960's, and earlier, have become the focus for inspection and evaluation of safety. Modifications have become necessary due to their age, and the application of evolving design criteria to the older structures.

1.3.2 Evolving Design Criteria and Regulatory Demands

Over the 5,000 years of dam development, it is clear that the means and methods of extending service life and improving performance have changed dramatically. Most of the important changes have occurred in the last 250 years

In 1853, M. de Sazilly published his method of analysis of gravity dams in a paper entitled "*Sur un type de profil d'egale resistance propose pour les murs des reservoir* *d'eau."*(Wegmann, 1911) Without revealing how, he concluded that the cross section in Figure 1.3 would be stable under all loading combinations up to a height of 50 m.

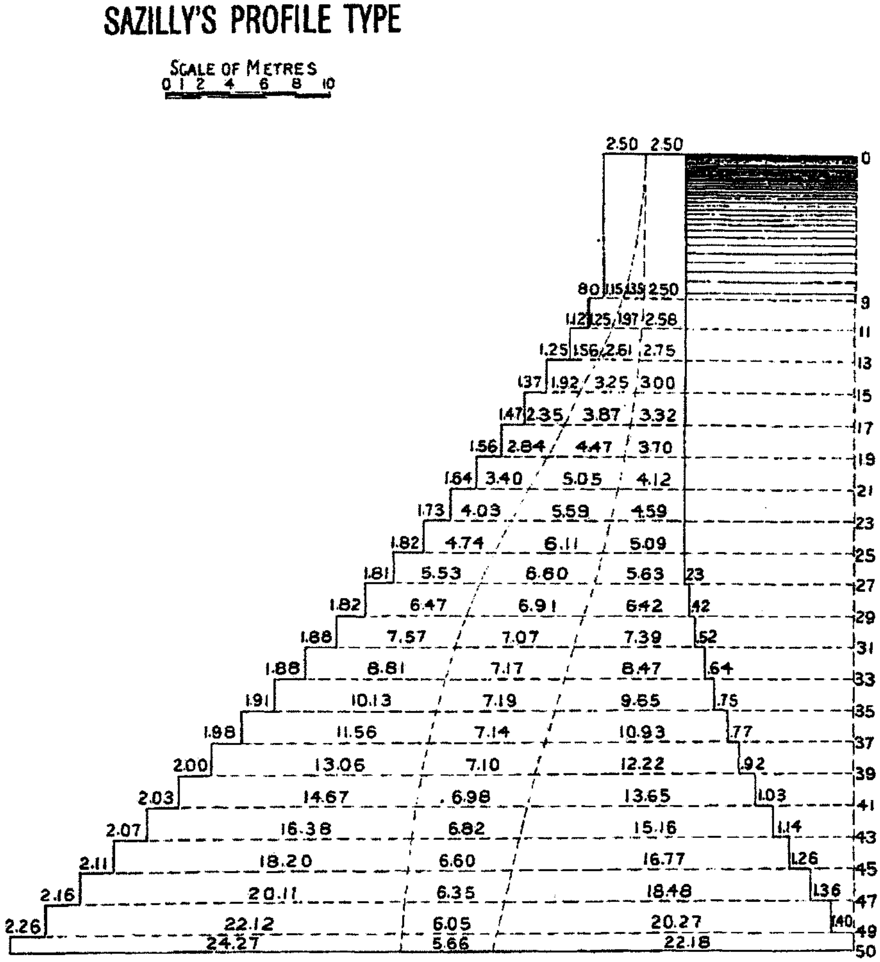


Figure 1.3 **de Sazilly's Profile (courtesy of Wegmann)**

Comparison of this example with the FERC normal load combination example in Figure 1.4 highlights the similarities in shape, and illustrates the change in understanding of how loads are applied. Improved understanding of the loading combinations that civil works must accommodate, and how they behave under those loads, have fueled advances in design. In many cases dam failures have also driven the advancements of design criteria, construction methods, and the analysis of dam performance, as initiated after the Buffalo Creek and Teton Dam failures in the 1970s.

CASE 1 - Usual Loading Combination
Normal Operating Condition

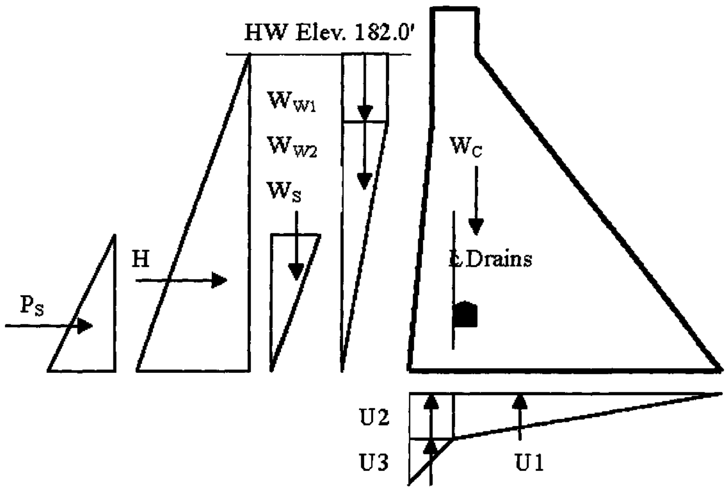


Figure 1.4 **FERC Sample Loading Diagram (courtesy of FERC)**

Two examples of evolving analytical techniques often affect planning for extending a project's service life:

- Recent advances in seismology and earthquake engineering may result in increased estimates of the design basis earthquake magnitudes, and resulting ground motions.
- Recent advances in meteorological and hydrological science and modeling techniques, may result in increased, or decreased, estimates of precipitation and resulting design basis floods.

Both of these advances have caused changes in regulatory criteria.

Modifications resulting from the incorporation of new knowledge into the safety evaluation of existing structures may be costly, yet are vitally important.

Often, changes in design philosophy serve to dictate the kinds of corrective measures that may be applied to extend service life. For example, the hydraulic fill method was commonly used for dam construction early in the 20th century. Experience has shown that embankment dams constructed by the hydraulic fill method are particularly susceptible to severe damage during strong ground shaking. Corrective measures invariably include strengthening and replacement of materials for hydraulic fill dams located in areas where strong ground shaking is possible.

Occasionally, changes in design philosophy lack an appropriate direct corrective measure. For example, many embankment dams were constructed before filter criteria were developed. This resulted in some dams having an elevated risk of piping through the core in comparison to embankment dams with appropriately designed filters and drains. Unfortunately, no appropriate direct corrective measure for this deficiency is possible because not enough of an embankment could be removed to place appropriate filters and drains. Consequently, other indirect corrective measures must be sought, such as constructing an inverted filter buttress to reduce piping risk, adding surveillance instrumentation, or lowering reservoir levels.

Chapter 2 and Appendix B further discuss the evolution of design philosophy, and how these changes relate to the project information required for good decision-making. Appendix A contains several references on current design criteria published by various federal and private organizations and societies.

1.4 Initiating Change

The spark that ignites the need to change may have its source within, or outside, the owner's control. The fundamental distinction between sources is easy to understand if internally initiated change is recognized as elective, and externally initiated change is seen as imposed.

An owner may elect to initiate change for safety or economic reasons. The obvious economic reasons are to increase revenue, or decrease expense. Opportunities for increased capacity or energy include increased head, reduced head losses, or more water for power. Increasing the forebay water elevation, adding a unit to take advantage of additional water, and smoothing conveyance lining can improve economics. Another approach to improving economics is to make changes that decrease operating costs. A typical example might be automating trashrack cleaning to reduce labor costs.

Safety is a paramount consideration for all owners, and owners initiate project improvements to improve public and employee safety.

Externally driven changes may be required because a regulatory agency, a non-governmental organization (NGO), or the public demands change as a result of periodic inspection or as a result of FERC licensing. The vast majority of non-Federal hydroelectric projects are licensed by FERC and, for licensed projects, demands for change often surface during the consultation process and are formalized in the conditions of license.

Regulatory demands for changes most commonly involve safety or environmental concerns. Safety concerns often stem from changes in estimated magnitudes of earthquakes and floods, and in downstream criteria or development. Damage or deterioration resulting from alkali-aggregate reaction, corrosion, cavitation, erosion, sedimentation, weathering, construction flaws, natural disasters (floods, earthquakes, and landslides), and human error may also raise safety concerns. Environmental concerns often relate to a project's impact on fish, wildlife, biota, recreation, aesthetics, and public interests.

Stakeholders may influence other imposed changes. The accepted definition of 'stakeholder' is anyone with an interest in a hydroelectric project. Stakeholders can include agencies with jurisdiction and expertise, such as FERC, US Fish and Wildlife Service (USFWS), National Marine Fisheries Service, agencies administering the Clean Water Act, United State Army Corp of Engineers (USACE), and the Environmental Protection Agency (EPA). Other stakeholders that may also influence how, and if, change occurs, are the Sierra Club, Audubon Society, Friends of the Earth, Nature Conservancy, and American Rivers. These NGO's routinely contribute to the environmental processes related to hydroelectric civil works and influence the outcome. Members of the public, not associated with an organized group, may also play a role in deciding whether change may occur.

1.4.1 Make It Last Longer and Work Better

What does all this mean in the context of improving performance and extending the service life of civil works? Change often follows the general process outlined in Figure 1.5. This figure attempts to summarize the process of a life extension or upgrade as captured in this guideline:

- The Drivers: (Internal and External Change) in the context of improved performance, extended life, regulatory and public demands for safety and environmental concerns.
- The Process: As summarized in Chapter 2 – namely information gathering, alternatives, and evaluation.
- The Experience: Has it been done before and what experiences and references are available (Chapters 3 – 7).

The keys to successful change lie in the answers to these questions:

- Why is change being considered? Is it an internal change to improve the performance or extend the service life? Or, is it an external change, driven from a regulatory demand (such as a FERC licensing or dam safety regulation or criteria) or a demand from the public for environmental performance? Or, is it a combination of both drivers?
- What are the opportunities and constraints to making a change?
- Do I have enough project information to understand the problem and its causes?
- Has this problem been solved before and can that knowledge be applied?

- What are the alternatives?
- Which is the best alternative to implement?

Figure 1.5 outlines the flow of this guideline, with the focus on making hydroelectric projects' civil works features *"last longer and work better."* The committee's approach to this objective was to briefly describe the decision-making process and innovative technologies (Chapters 2-3), and in the balance of the book (Chapters 4-7), examine the function, problems, corrective measures, opportunities, case histories, and references for a variety of hydroelectric component structures.

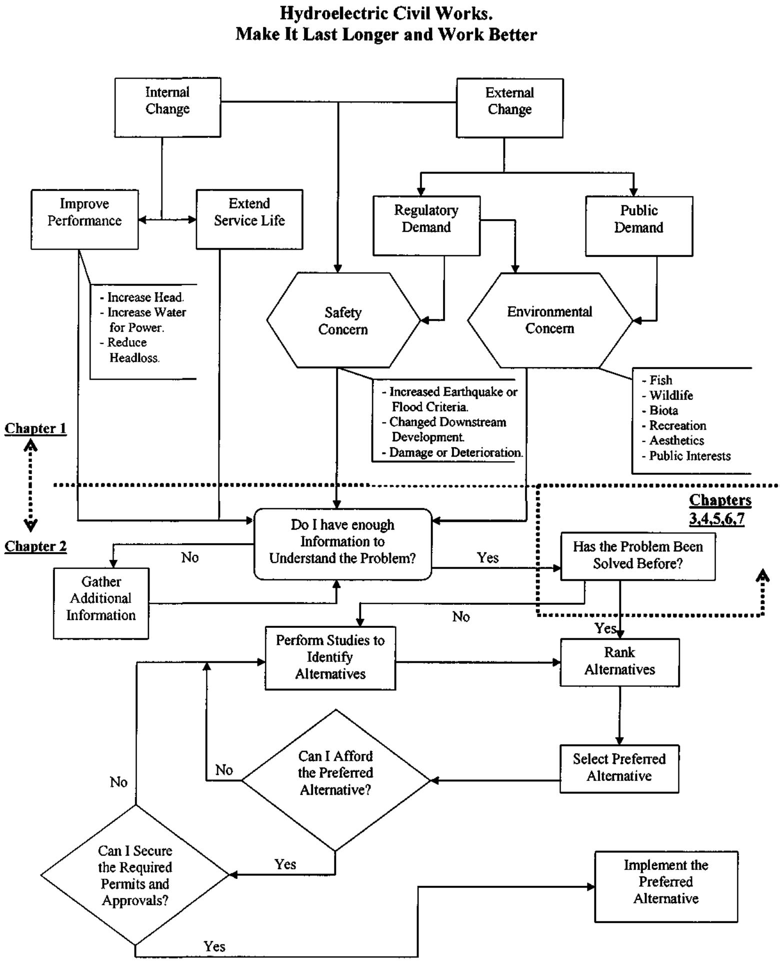


Figure 1.5 Process of Change
(courtesy of Committee Collective Knowledge)

1.5 Technical References

American Society of Civil Engineers (ASCE). (1992). *Guidelines for Rehabilitation of Civil Works of Hydroelectric Plants.* ASCE, New York, NY.

Federal Energy Regulatory Commission (FERC). Office of Hydropower Licensing. (1991). *Engineering Guidelines for the Evaluation of Hydropower Projects*, FERC, Washington, DC.

United States Army Corps of Engineers (USACE). (1996). *National Inventory of Water Control Structures.* Portland District, United States Army Corps of Engineers.

Wegmann, Edward. (1911). *The Design and Construction of Dams.* John Wiley and Sons, London.

1.6 Collective Knowledge

American Society of Civil Engineers (ASCE). Task Committee on Civil Works for Hydroelectric Facilities: Guidelines for Life Extension and Upgrade. (2003).

2.0 *CHAPTER 2 – PLANNING AND EVALUATION*

2.1 Introduction

The purpose of this chapter is to discuss the planning of the life extension and upgrade of the civil works components of a hydroelectric project. While these components are essential, the electrical/mechanical aspects are additionally important in the total plant upgrade. Further information on planning and evaluation of total power plant life extension or upgrade can be obtained from EPRI (EPRI, 1999). The civil works life extension projects must be combined with the consideration of the electrical/mechanical aspects to provide the total picture of a plant upgrade.

However, there are instances where a civil works feature needs attention, without consideration of total plant improvement. In this case, this guideline is particularly worthy, since it focuses on component features as individual "projects." Identification of need for the project, requirements for accurate data and condition assessment, development of alternatives, and selection of the preferred course of action are examined. Notes on regulatory considerations, cost estimating, constructability, staging and scheduling, access and contracting options are included.

Examining the hydroelectrical project as a collection of component features allows for great detail and reference to excellent sources of technical information and processes already available to the practicing engineer. Information on life-cycle analysis, condition assessment, and synthesis of data as collected is available through the previous ASCE hydropower committee reports (ASCE,1992). The goal here is to outline a general process of evaluation of the components, as an introduction to the experiences contained in Chapters 4-7.

2.2 Planning and Evaluation Process

The planning process, as depicted in Figure 1.5 in Chapter 1, consists of the following steps:

- Identify need for project.
- Assemble data and information and assess condition.
- Identify parties involved.
- Identify, plan and evaluate alternative courses of action.
- Compare alternatives and select a preferred course of action.
- Proceed with a life extension or upgrade project.

The outcome of the planning process, unless the decision is to take no action, or no immediate action, will normally result in the implementation of a project, subject to any priority or financial constraints. Implementation commences with the preparation of specifications, drawings, cost estimates, construction schedules, and the selection of appropriate contracting methods.

The level of effort, and the extent of the planning and evaluation required in each case, varies with the owner, the type and size of the project, the cost involved, project benefits (such as cost reductions or revenue enhancement), and the amount of information to be collected to make a decision. Some projects have to be economically justified. Some projects are mandatory. Some projects have to meet budgetary requirements. Whatever the circumstances, the basic process is the same, with different emphasis and effort being placed on each step.

2.3 Identify Need for Project

The need for extending service life, or for improving the performance of structures, or other civil works, of hydroelectric projects, may be driven by mandatory or discretionary reasons. In some cases these needs may overlap.

2.3.1 Mandatory Actions

Mandatory actions are those required by prudent practice, or by regulating agencies, relating to such items as safety, operations, and environment. If more than one way of satisfying these actions exists, then the preferred course of action typically takes into account all of the engineering, environmental, and economic factors, including cost.

In some cases, emergency actions to preserve the safety and operation of hydroelectric projects are required, and are therefore considered a form of mandatory action. These actions may be the result of unsafe conditions from aging, damage caused by extreme events, or human error. Problems at aging facilities include deteriorating concrete, corrosion, equipment failure, obsolescence, and structure that do not meet current design criteria.

Relicensing may also introduce mandatory action items. As part of the relicensing process, the condition of the project is assessed to verify that it will perform satisfactorily and safely for the life of a new license. In addition, relicensing may impose conditions to address new environmental concerns, or identify more efficient ways to utilize project resources.

2.3.2 Discretionary Actions

Discretionary (voluntary) actions are those within the control of the owner. Such actions usually are implemented only if they benefit the owner, and may include actions that improve safety and performance, reduce operating costs, increase net income, or extend service life.

Net income may increase when modifications increase the amount of water available for power generation, reduce head losses, or reduce operation and maintenance costs for a given project. For example, installing or applying a new coating on the inside of a penstock may reduce head loss and thereby increase power output. Other examples are dam raises which increase the available head, and the installation of a pneumatic flashboard system which can increase annual generation and reduce operation and maintenance costs.

2.4 Assemble Data and Information and Assess Condition

Good decision-making doesn't happen in a vacuum. Rather, it relies on adequate information. *Informed decision-making* might be a better term. This section discusses the information necessary to evaluate civil works' life extension and upgrade projects under the following headings:

- Sources of Information – Owner and Designer, Regulators, and Project Personnel Knowledge.
- Condition Assessment.
- Beyond Engineering - Non-Technical Data.

The life cycle of hydroelectric civil works generally follows the following sequence:

- Planning, investigation, design, bidding, construction and contract closeout.
- Testing, operation and maintenance.
- Consideration of life extensions or upgrades.

Each civil work's service life began with an idea translated into design and supported by investigations that may have been brief, extensive, or something in between. The degree of thoroughness notwithstanding, investigations provided enough information to design the civil works. Operation and maintenance continue throughout the life of a project and associated records provide information on the performance and limitations of the feature.

At the stage when an owner begins to consider extending the service life, or improving the performance of a civil work, the question is, "What information is needed to promote informed decision-making?" The answer depends on what is proposed, and there is no simple formula for adequate information gathering. Gather whatever information is required for informed decision-making. In simple terms, the information might follow the basic elements of any engineering feasibility study—the collection of the technical, environmental, institutional, legal, and economic/financial information about the project.

Each of the basic elements has its own merit. Technical information describes the design, bidding, construction, and operation of the civil works feature. Environmental information demonstrates that the project was compliant with the environmental regulations during its lifetime. Legal feasibility is important because the project has the legal right (property ownership, water rights, and other legal issues) to be constructed and operate. Institutional requirements are met if all permits and approvals to construct and operate the project were secured. And finally, the traditional yardstick of adequate return on investment is met by economic feasibility, as well as the ability to secure adequate funding to assure financial feasibility.

2.4.1 Sources of Information

Where is the information likely to reside? The most potentially useful sources are the project files maintained by (1) the original owner and designer, (2) regulators, (3) current owner project personnel, and (4) local libraries, and historical associations. Local interviews, including, where possible, workers involved in the original project construction are also possible sources.

Owner and Designer

Owners' and designers' project files are likely to contain the most complete project records. Table 2.1 summarizes project information likely to be available. The types of information shown in the table are intended to provide guidance, and may not be complete, depending on each individual project's circumstances. It would be extraordinary to consider a project with no prior information. That said, many projects have poor or incomplete records. It is not unusual to find that a project has no record of design basis or as-built drawings.

Table 2.1 Project Information

	TECHNICAL AND OPERATING	ENVIRONMENTAL AND INSTITUTIONAL	LEGAL, ECONOMIC, AND FINANCIAL
Design	• Design basis documents (mapping, preliminary layouts, site investigations, material properties, hydrology studies, hydraulic design, design criteria, design calculations, drawings, specifications, and contract documents).	• Environmental studies (assessments, biological opinions, impact statements, and checklists).	• Property acquisition and sale records. • Deeds. • Right-of-ways and easements. • Certificates of water rights. • Records of project related litigation.

	TECHNICAL AND OPERATING	ENVIRONMENTAL AND INSTITUTIONAL	LEGAL, ECONOMIC, AND FINANCIAL
Construction and Operations	• Construction records (material testing, change orders, photos, progress schedules, unforeseen conditions, and claims). • Operating records (changes made to civil works and the reasons for the changes, performance monitoring records from instruments and measurements, safety and operational inspection reports, and records of performance during extreme events such as floods and earthquakes). • Records of start-up testing and any subsequent in-service testing.	• Project permitting and licensing history, or exemptions with all applicable conditions. • Water quality certification. • Dredge and fill permits. • Hydraulic project approvals. • Reservoir filling approval and operating limitations. • Recreational requirements. • Minimum stream flow requirements. • Fish and wildlife mitigation requirements. • Records of environmental operating compliance and violations.	• Economic feasibility studies. • Annual operating budgets and expenditures for maintenance. • Capital versus Operating expenses. • Project revenues and cash flow. • Debt service, taxes and insurance, and annual payments for water rights, headwater benefits and other.
Maintenance	• Maintenance records (descriptions of major [non-routine] maintenance work, including why major maintenance was required, and how it was performed).	• Emergency Action Plans. • Stakeholder groups and activities.	• Financial history, including project financing and restrictions. • Power purchase agreements and negotiations. • Power marketing, wheeling and transmission agreements. • Future power projects and market assumptions.

(Committee Collective Knowledge, 2005)

Regulators

Almost without exception, all hydroelectric projects receive some regulatory oversight. Federal projects operated by the Bureau of Reclamation, Corps of Engineers, and the Tennessee Valley Authority essentially are self-regulated. Non-federal projects, both public and private, are regulated by FERC. [FERC maintains central records for all licensed projects in a system called FERRIS (Federal Regulatory Record Information System), a successor to the system known as RIMS. Access to the FERRIS system is available at the FERC's website and is limited because of security concerns. However, in good time, any record may be retrieved from FERRIS under the Freedom of Information Act (FOIA). The few projects that escape Federal regulatory oversight are State-regulated.]

States may maintain an agency or department to address dam safety, and a variety of record-keeping systems may be employed. An example of a state agency with perhaps the most complete record keeping is the California's Division of Safety of Dams (DSOD) system.

In 2003, FERC launched an initiative requiring a Potential Failure Modes Analysis (PFMA) for licensed projects. The PFMA complements the 5-year safety inspections required under Part 12, Subpart D of the FERC regulations. To support the PFMA, FERC requires each licensee to compile a Supporting Technical Information Document (STID) containing at a minimum:

- Project description and drawings.
- Construction history.
- Geologic and seismic assessment.
- Hydrology assessment.
- Spillway adequacy.
- Stability assessments of water-retaining structures.
- Status of studies in process and outstanding issues.
- References.

Each licensee is required to maintain the STID current. In the context of extending the service life, or improving the performance of a project's civil works, the STID can be a rich source of useful information.

Project Personnel Knowledge

The above discussion centers on paper or electronic records – calculations, test results, drawings, photographs, reports, and the like. Often overlooked in the search for information are people – operators, project engineers, and maintenance crews. These people invariably have information not available on paper, or on computers, personal experience that may be key to informed decision-making. Imagine these clips of conversation:

- "The spillway doesn't have stoplog slots. How did we get access to paint the upstream face of the skin plates on the gates?"
- "Are divers required, and how long does it take to install the draft tube gates?"
- "Why did the RCC buttress costs overrun the contract price?"
- "When were the Lubrite bearings installed?"
- "Why did we stop reading the strain meter in Block 24?"

Capturing this important, undocumented project knowledge has become increasingly vital as the workforce ages and retires and the energy industry and hydroelectric owners have initiated significant programs of knowledge "mining" and transfer. In collecting project information for a life extension or upgrade project, this type of operating information can be key. The following serves as a possible guide to eliciting such information.

- Describe any special geographic information you may have about where things are located, and how to get to particular locations, including the easiest way to get there.
- Describe any special information you may have about the location or existence of spares, materials, tools, and equipment.
- Describe any special information you may have about key contacts for expert advise, decisions, permissions, getting something processed?
- Describe any special information you may have about where to locate maps, lists, drawings, vendor manuals, design data, calculations, start-up reports.
- Describe any special information you may have about how to order parts, materials or services, or where to get equipment repaired or calibrated.
- Describe or list any non-standard knowledge that you possess, or have developed, about the diagnosis of complex problems.
- Describe or list any special knowledge that you may have about specific pieces of equipment or structures that would lead to rapid diagnosis of failure.
- Describe or list any special knowledge you think you may have about patterns of equipment or structural performance deterioration that predict major failures.
- Describe or list any special knowledge you may have about failure patterns for particular pieces of equipment or structures that would lead you to do preemptive inspection or replacement.
- Describe any historical knowledge (lessons learned) you have that might help avoid the repeat of a major error in the future.
- Describe any special hazards that may exist.

Local Libraries, Historical Associations and Local Interviews

Local libraries can be a valuable source for project information. Generally, libraries will maintain a section on local history containing newspaper articles, journals and other documents discussing the historical accounts of hydroelectric construction. Old photographs can sometimes be found showing construction work in progress. Local historical associations will also maintain period documents that can be helpful in determining factual accounts of the project. In some instances, interviews with local people, such as historians and genealogists, can lead to hydroelectric related documents residing in people's homes. These can include descendants of former hydroelectric plant personnel, and others who were involved with the original construction or modification of the facility, who have kept photographs, diaries and other records that could be valuable information to engineers and owners considering life extension or upgrades.

One example of this type of source involved a 1.5MW hydroelectric facility with a 60 foot high earth dam built around 1905. Very little technical information was available on the original construction of the dam. Based on what little information existed, it was believed a concrete core wall was included in the original dam construction, but no definitive evidence existed. It was not until an interview was conducted with the town historian, that an old newspaper article was discovered with a construction photograph showing the concrete core wall being built. This piece of information was helpful in planning the installation of observation wells within the dam and saved several thousands of dollars in investigative costs.

2.4.2 Condition Assessment

In a previous document prepared by an ASCE Task committee (ASCE, 1992), the focus was on: "describing the methods for assessing the condition of the civil works of hydroelectric plants and to outline some of the procedures available for civil works rehabilitation and repair." A brief synopsis of that document follows (It should be noted that the ASCE, 1992 document covers powerhouse and water conveyance/control features only, and not dams, spillways and related structures):

Chapter 2 – Site Characterization outlined some of the information necessary to complete a condition assessment—including the materials and design standards used in the original design, the initial operational objectives, and the changes in operation of the facilities throughout the years.

Chapter 3 – Inspection and Assessment provided a fairly detailed inspection checklist for various civil works structures, including examinations, instrumentation, and non-destructive testing techniques.

Chapter 4 – Rehabilitation and Repair Procedures discussed some of the techniques available for civil works repair, drawing largely on the ongoing USACE REMR Program (USACE, 1984).

Chapter 5 – Inspection and Maintenance Manual contains a suggested program for an ongoing I & M program for civil works of hydroelectric plants including start-up, normal operating conditions, high/low flow conditions, emergency operations, maintenance plans, inspections and record keeping.

Another document that is useful in assessing the condition of hydroelectric civil works is the *Safety Evaluation of Existing Dams* manual, known in the industry as the SEEDS Manual (USBR, 1980). This document provides engineering guidelines for the inspection and evaluation of dams. Topics discussed include the purpose of a safety evaluation, modes and causes of failures, the evaluation team, purpose of the project data book, evaluation of the design, construction and operations of the dam, the field examination and final reporting.

Summarizing the condition of each of the project features, based on the data collected, is usually the final step in the preparation for an upgrade or rehabilitation project.

There are several approaches to evaluating the overall condition, first, of the components, and then, of the total overall project. The evaluator should exercise the same level of step-by-step documentation, inspection, and testing. In evaluating the costs for repair of each component, a similar estimate of the value of benefit of the repair should be added either empirically or subjectively.

In arriving at the condition of a project feature, the evaluator considers the design criteria, an assessment of the remaining useful life, and any new examinations or tests that would aid in quantifying the life extension study. An aspect to be addressed is whether the structure has the ability, or capability of meeting, its intended role or purpose without posing an unacceptable risk. Condition assessment also includes the estimation of costs for isolating and handling hazardous waste materials and environmental mitigations and improvements. While each project is different, a condition assessment scheme is suggested to assist the evaluator in reflecting the evaluation for each project feature or component. In general, as the condition rating of a structure decreases (i.e. The structure is becoming more deteriorated), then the operability and maintenance rating of the structure decreases, the remaining service life of the structure decreases and the cost to life extend the structure increases.

While this simplistic view of condition assessment cannot capture the nuances associated with cases where a replacement (particularly with a new material or technique) may be more cost effective than a complicated repair, it demonstrates the intent to systematically collect project information, and assess project feature condition and operability, in order to summarize the total life extension or upgrade project.

2.4.3 Beyond Engineering

The discussion above focuses on collecting and analyzing largely technical and engineering information. However, being able to jump the technical, environmental, legal, and institutional hurdles is not enough to move a proposed service life extension or improvement project through the often-tangled web of management. The proposal must also make economic and financial sense.

Specific types of data are needed for the purpose of performing economic assessments (sometimes referred to as an operational assessment). This may be required for a number of reasons. These include potential sale, due diligence for potential purchase, refinancing, evaluating insurance coverage, or benchmarking for performance assessment when considering relative economics of life extension and/or upgrade options (i.e. Does it make good economic sense to pursue a particular course of action?). Where required, an economic assessment fundamentally requires an estimate of annual revenues (benefits) and annual costs. This information is then fed into various formulas to determine economic viability such as Net Present Value, Benefit-Cost or Internal Rate of Return.

These assessments require certain data to support informed decision-making. In addition to information already described above, relevant data may also include the following:

- Hydrologic data – stream gauge and flow duration information, and an understanding of compliance requirements.
- Annual revenues from historical energy production, which includes both the efficiency of the units (as analyzed from commissioning and index tests) and the power sales contracts which outlines revenue from capacity and energy as well as an understanding of ancillary services markets now and in the future. It is important to understand changes that have occurred over time for example to unit efficiency or to head losses.
- Annual operation and maintenance costs including major maintenance and capital expenditure records for at least the past 5 to 10 years and future projections.
- Project Economics—the overall situation and assessment of all information.

Hydrologic Data

Fundamental to any hydroelectric generating project is the flow available for generation. Typically, flow data is available from reliable sources such as the USGS, or from physical flow measurements by the hydroplant owner. When direct river flow data is not available, flow data from an adjacent drainage basin may be used with appropriate adjustment factors. Historical project generation should be correlated against historical flow available for generation. Long-term data is more statistically relevant, provided that no changes in upstream regulation have occurred which might impact the project. Ask the question: "Are potential changes in upstream regulation planned or contemplated?" As in the stock market, "past performance is no guarantee of future returns." Also, hydrological data can be affected by land use changes, both upstream and downstream of the project.

Annual Revenues

For many hydroelectric facilities, annual revenues are primarily derived from energy sales. While there are revenues from capacity and ancillary services such as black start, regulation and voltage control, this section will focus more on the energy estimates needed when performing economic assessments.

When determining expected future generation levels, records of historical generation levels are used as a basis to estimate future levels. Historical generation must be evaluated with care. If there is a significant deviation between historical generation and average expected generation in any given year, further research is required before concluding that the variation is due to hydrologic conditions. Past operation and maintenance records must be examined to determine if planned or unscheduled outages significantly affected the annual generation. Generation data on an hourly basis may be required depending upon individual project circumstances. Many larger projects operate on a store and release basis to maximize the value of the energy produced, shifting generation from off peak periods to the higher value peak periods of the day or week.

Operation in the de-regulated market of the late 1990's and early 21st century may differ from historical operation by regulated utilities. Historical energy production may not be reflective of future generation, especially in a market driven environment. Changes in operation can have a significant impact on project economics beyond the obvious. For example, many utilities operate units as synchronous condensers during periods of non-generation. In a de-regulated market, an Independent Power Producer (IPP) may shut the same unit down to avoid the expense for motoring energy if there is little, or no economic benefit, for producing reactive power. Before deregulation, many utilities would strive to maximize their annual energy production regardless of the time of day. In a deregulated environment, IPP's are more focused on maximizing revenues, not necessarily generation. Care must be given when extrapolating historic generation values to estimate future annual generation levels.

Annual Costs

Depending upon the reasons for the project assessment, certain annual costs should be determined and reviewed. These historic annual costs include FERC fees, property taxes, insurance, operating expenses, maintenance expenditures, capital expenditures, and financing costs (i.e. cost of capital). Once these historic expenditures are known, this information can be used in developing estimates of future expenditures, incorporating any changes in operations as appropriate. If ownership and operation is to be transferred, certain annual cost structures may change. These costs can include annual FERC fees, property taxes, and insurance.

For example, different FERC fee structures exist for municipal owners. Some organizations are exempt from property tax payments. Others may be eligible for a PILOT agreement (Payment in Lieu of Property Tax). Depending on what state the facility resides, property tax reduction programs may exist that can significantly impact the amount of tax a facility owner is obligated to pay. Some organizations are self-insured or benefit from a larger portfolio of projects. Insurance coverage should be reviewed periodically. Typically, property insurance is based upon replacement cost. Many project managers will assess current replacement costs against the level of insurance coverage and the owner's tolerance for risk. Business interruption insurance is typically written based upon expected energy production. Insurance policies should be reviewed if changes in operation occur or if expected generation changes. If there is a change from private ownership of a project to public ownership or vice versa, there may be changes in interest/discount rates.

The age and condition of project components has a direct bearing on future operation and maintenance costs. As hydroelectric components age, maintenance requirements normally increase to ensure the same level of reliability. Unit performance and efficiencies decline with age. It is important to understand the original project design criteria, current mode of operation versus the original mode, if different, and maintenance history for major components (i.e. Is the turbine runner original? Has it been replaced? If so, more than once?). The useful remaining life is an important, but often difficult, parameter to define. The number of years of estimated remaining useful life has significant impacts on project economics. For example, a twenty year economic analysis for a facility requiring no work on the dam during that period will be economically stronger than a project where the dam has reached the end of its useful life, requiring a major capital expenditure within the twenty year period. Major maintenance expenditures must be budgeted appropriately for the project to perform according to pro-forma projections.

Construction history can also be an important consideration. Often, a project is constructed with certain features that were never intended to be permanent. Examples include intake openings constructed for future use that were sealed with temporary timber bulkheads. As years pass without any activity at these locations, these components can easily be forgotten, only to surface as considerable problem areas many years later, often to the surprise of the owner. Construction records can also be valuable when budgeting for future maintenance. Intakes or water passages may not have been constructed with stop log or bulkhead slots, which would necessitate more costly means of dewatering. Powerhouse cranes may not have sufficient capacity or headroom to remove equipment. Review of design and construction details in the budgeting process for major work can provide very valuable information.

Project Economics

Trending certain performance and cost data can be very worthwhile. Tracking and plotting the cost of O&M on a dollar per kilowatt-hour basis over time can point out the need for repairs or replacements, as well as identify the need for greater scrutiny of individual cost accounts.

Economic performance can be evaluated after all costs and benefits (i.e. Revenues) have been identified. Project revenues are heavily dependent upon generation, capacity, and ancillary services, type and timing of project operation, and the type of markets the project participates in. If projects are to be evaluated on an individual basis, some costs may have to be allocated to each project if they are part of a larger portfolio of projects, or share overhead and operational cost structures. Administrative costs, insurance costs, O&M costs and the cost of spare parts must be allocated to individual projects, and properly compared to project benefits, in order to compare project performance against financial benchmarks such as Internal Rate of Return (IRR), Return on Investment (ROI) or Net Present Value (NPV).

- To ensure that energy production estimates are not unrealistically high, include proper headwater and tail water curves; head losses at gates, trash racks and draft tube; losses at speed increasers, generators, and transformers; scheduled and unscheduled outages; flow data representing wet or dry periods of time; and flow reductions for leakage, withdrawals, and environmental releases.

The value of the power should be defined for the life of the project by identifying the present market value, trends for the next 3-5 years based on marginal cost values (i.e. gas costs), and then beyond 5 years, by using a reasonable escalation rate for the value of the power over the life of the project.

- Verify that cost estimates are complete, and include such items as land acquisition, interconnection equipment, transportation and installation of equipment, and environmental mitigation.

2.5 Identify Parties Involved

In planning life extension and upgrade projects, there are four main parties involved, as follows:

- Owners - Project owners include both public and private entities. Public entities encompass irrigation districts, municipal and public utilities, state and federal agencies. Private entities include private developers and independent power producers. Owners typically arrange for the planning and execution of projects. This covers planning studies, meeting license and permit requirements, and ultimately, design, construction, commissioning and operation.
- Regulators – Federal entities are generally self-regulated. Non-federal entities are regulated by the FERC. To fulfill license requirements, non-federal entities may require comments from agencies, and certain permits, principally the Section 404, Dredge and Fill Permit from the USACE and the Section 401, Water Quality Permit from the USEPA.
- Agencies – At the federal level, typical agencies are USFWS, USEPA, USACE, USFS and NOAA. State departments of conservation and environmental protection, and Indian tribes, also fall into this category. The appropriate state dam safety agencies will be involved at non-federal sites. Agencies typically review license applications and certain compliance items, and as indicated above under Regulators, issue permits to fulfill license requirements.
- Public - The public is a group of interested parties and is comprised of rate payers (of electric utilities), users of the project's water resources and associated recreational and developed lands, property owners, and non-governmental

organizations (NGOs). The public can input to the regulatory process through document review and attendance at public meetings. It should be noted that the interest of electric utility customers (rate-payers) will often be different from that of the public impacted by the project.

2.6 Plan and Evaluate Alternatives

While companies and organizations approach and evaluate situations differently, there are some key elements common to the process of identifying and evaluating alternatives, such as:

- Regulatory considerations.
- Cost estimates.
- Constructability.
- Staging and scheduling.
- Access.
- Contracting options.

2.6.1 Regulatory Considerations

Securing construction permits and approvals from local, state, and federal regulatory agencies can constrain actions to extend service life or improve performance. Permits are often needed to increase or reduce flow releases into the river, provide access to the area of work, for environmental protection, for the construction of cofferdams, and for the construction project itself.

Regulatory requirements imposed by Federal, state, and local governments or those resulting from public relations issues include:

- Federal and state requirements:
    - Historical and archaeological
    - Architectural
    - Section 404, Dredge and Fill Permit
    - Section 401, Water Quality Permit
    - Endangered Species Act (ESA)
    - Environmental Impact Assessment (EIA)
    - NEPA process
    - Discharge permits
- Local building and other requirements
- Public relations and communications considerations:
    - Native lands
    - Religious
    - Local customs
    - Recreational

2.6.2 Cost Estimates

Costs should be estimated for each of the alternatives to aid in determining the best option. At this stage, the estimates may not have a high degree of accuracy, but will help the engineer determine the order of magnitude of difference between alternatives.

Later, after the preferred alternative is identified and final design is complete, an estimate should be done to determine the cost of the project. The estimate will be used for internal budget and to help evaluate the contractors' bids as they may vary significantly. The estimate will help determine if the contractors have understood the scope of the work, or if further clarifications are required.

Site-specific parameters need to be taken into consideration in the cost estimate for a project. These include items such as the local wage rate, and availability of labor. Labor rates must reflect whether union or nonunion labor is used, and if adequate skilled labor is locally available. Labor is often the largest cost component of a project, and any increase in labor costs will add significantly to the overall project cost.

Based on the location of the project, skilled labor may not be readily available. Finding skilled labor can be a problem in urban areas with low unemployment, or in rural areas. This can have a significant impact on the cost and schedule. The contractor may have to pay a premium to bring in outside labor to maintain quality workmanship. The availability of material and equipment also needs to be factored into the cost, as does the cost of delivery to the site. In remote areas, some building materials, such as ready mixed concrete, may not be readily available, necessitating the installation of a batch plant, which adds both to the cost and schedule of the project. Other materials may need to be trucked in from long distances, requiring longer lead times and higher costs.

Generally, most large equipment used by contractors is rented for a particular project. In remote areas where equipment must be brought in, rental costs may be higher, as the contractor must rent the equipment for a longer period of time in order to mobilize, and demobilize, from the remote site.

2.6.3 Constructability

In addition to the cost estimate, a constructability review should be done to ensure that the details of constructing the project have been well thought out. This includes items such as access to the work area, availability of material, labor, equipment, and methods of construction.

Special consideration should be given to locating large equipment, such as cranes, to be able to reach the inaccessible areas. Areas such as the toe of the spillway can be a challenge to access for placing and removing equipment and materials.

A determination of any required permits to do the work will be necessary. For example, a long lead-time may be required to get approval for a permit for activities such as a reservoir drawdown, or for work in a waterway.

An important aspect of the life extension or upgrade of hydroelectric plant civil works is the ability to carry out the construction work in dry conditions. In the case of the power plant, construction work may be able to proceed by closing intake and draft tube gates. Individual unit bays can be unwatered by this method. However, work at canal head gate structures, power plant intakes, tailrace areas, and other watered areas may require some form of cofferdam. Dewatering and the provision for cofferdams are significant cost and schedule items, as well as permitting and environmental items.

Typically, older power plants utilized some materials, which, by today's standards and regulations, are hazardous to the environment and humans. Assessment of the presence of these materials is important because such hazardous materials may have to be encapsulated or removed prior to commencement of rehabilitation work. This can be a significant cost and schedule item. Such materials include lead paint and asbestos insulation.

2.6.4 Staging and Scheduling

An important pre-construction activity, and one that will likely be a large part of the constructability review, is the determination of the work sequence and duration. Factors to be considered include the impact of construction activities on generation, recreational activities, fish and wildlife. Generally, work should be scheduled during periods of low flow, or during times of low value of power, such as off-peak periods, to minimize impact on revenue. It should also be sequenced to keep as many units operational as possible. Work in the spillway area needs to be staged, since, in the event of a large storm, an adequate number of gates must be available to maintain the required head pond elevation. In addition, the contractor must be able to remove large equipment in the spillway area in a relatively short time in the event that the spillway gates need to open. Staging of work must be decided up front so that any approvals required by state or federal agencies can be obtained with sufficient lead-time.

Construction in the waterway is usually deferred seasonally when flow in the river has reduced, and after typical fish spawning periods. In northern climates, the construction season may be limited to only five or six months of the year due to weather constraints. For large projects, the work may need to be scheduled over more than one year.

Getting approval to draw down reservoirs is often difficult. If approval is given, it is generally only for a very limited period of time to minimize any impact on recreation, fish and wildlife, and water supply. Staging and scheduling the work in areas requiring a draw down can be critical if a large amount of work is to be done in a short period of time.

2.6.5 Access

Hydroelectric plants located in remote areas may have difficult access and bringing in large equipment can be a problem. At older facilities, plant access roads may need to be upgraded prior to starting construction. If suitable access to a plant cannot be provided by road, other means should be investigated, such as barge or helicopter.

2.6.6 Contracting Options

Several different contracting options can be used when bidding a project. How well the scope of work can be defined, and how much involvement the state or federal agencies have with the project, will determine the best method to use.

- Design-Bid-Build - The traditional design-bid-build method of contracting is the most common. This method separates the design and construction phases of the work. It also allows the owner to build his own quality standards into the design instead of using the cheapest components. Once the design has been completed, the project is then bid out to contractors. The benefit of this method is that it allows the owner time between the design and construction phase to submit the design to the appropriate agencies for review and approval, and to incorporate their comments, before bidding the construction phase of the work and obtaining the appropriate permits. The agencies' comments can often change the design and approach to the scope of work. This method allows the opportunity to better define the scope of work prior to bidding the construction.
- Partnering - Partnering is a method where the owner takes some of the risk involved with construction. The contractor can lower his bid price by identifying those "extra" costs associated with certain work tasks, that are difficult to quantify, of high risk, or unknown. In partnering, the owner takes on the burden of the extra costs only if they materialize, or saves if conditions do no incur extra costs.
- Performance Based - Performance based contracts give the contractor the opportunity to make additional profit based on reaching milestone dates in the contract and possibly losing money if the dates are not met. This is generally used when there is some financial gain for the dam owner to have the project expedited, such as the need for generation, or when strict regulatory schedule constraints exist. This type of contract is usually a target estimate plus fee type, but can be a time and materials contract.
- Design-Build - Design-build method means a single contractor provides both the design and the construction. The contractor may choose to do both, or partner with an engineering firm to provide a complete package. This method works well for new projects where work can be clearly defined. On projects which have heavy involvement by federal or state agencies, it may often be hard to have a clear scope of work at the beginning of a project.
- Specialty Contracts - Some work may require specialty contractors, such as for underwater construction, where the general terms and conditions of the contract

may need to be modified to take into account special requirements like the type of insurance coverage needed.

2.7 Selection of Preferred Course of Action

2.7.1 Comparison of Alternatives

The decision on which alternative to pursue or abandon is normally based upon technical, environmental, economic and institutional issues. The most critical elements that affect life extension and upgrade opportunities for hydroelectric civil works are:

- The performance and safety of the dam.
- Risks and costs associated with doing nothing.
- Cost estimates as related to investment requirements and projected returns.
- Economic justification based on benefit/cost and repayment costs.
- Environmental considerations for both the short and long term impacts.
- Social or quality of life benefits.

Different methods will be used to evaluate alternatives for a hydroelectric facility civil works life extension or upgrade project. For mandatory items, it is likely that cost and schedule considerations will govern. For discretionary actions, payback period or rate of return may govern, or reduction in operation and maintenance costs.

Life extension projects are often evaluated on the basis of improved reliability, decreased operation and maintenance costs and improved output.

A factor to be considered in comparing alternatives is the cost of decommissioning. For some projects where life extension is uneconomic relative to anticipated revenues, rehabilitation may still be more cost effective than decommissioning. The decommissioning studies required can be extensive and costly. Further information on decommissioning can be obtained from a previous guideline document prepared by an ASCE Task Committee (ASCE, 1997).

2.7.2 Economic Comparisons

Economic comparison of alternatives can be made on the basis of:

- Capital costs.
- Lowest capital cost and the shortest payback.
- Rate of return, present value of net benefits or benefit cost ratios.

It should be noted that the capital costs may potentially include the costs for decommissioning (removal) of the project structures.

The payback period is the number of years required for the net operating revenues or benefits of a project to pay back the initial capital costs of the project. The payback may be calculated either as a simple payback period without discounting any future benefits or costs, or the discounted payback period. The discounted payback period is the number of years required after project commissioning for the discounted benefits of a project to pay back the discounted costs.

The payback period is a simple criterion to use – the lower the number of years it takes to pay back, the better the project. However, it ignores such important factors as the project lifetime costs, and the operating cost of a project beyond the last year of the payback period.

Another important economic comparison of alternatives is the internal rate of return (IRR) offered by each. This is simply defined as the rate of interest of money necessary to make the net present value equal to zero. This is a single rate of return that summarizes the project and is independent of other rates that might be offered. A standard criterion for an IRR comparison/analysis is to accept all projects with an internal rate of return greater than the opportunity cost of capital. Many private owners use this method with minimum required IRR thresholds of 12 – 15 percent prior to receiving approval for funding.

Net present value analysis evaluates all costs, expenses, and revenues discounted to present time dollars based on current interest rates. The breakeven point is the number of years it takes to make the discounted revenues, minus discounted costs (net present value) equal to zero. The following criteria represent some owner's perspectives with respect to net present value estimations:

- If the net present value is greater than zero, then accept the project.
- If the net present value is less than zero, then reject the project.
- If the net present value is equal to zero, then consider the non-cost issues to make a decision.

These criteria obviously vary among owners and projects. For example, if the analysis has a high degree of uncertainty with key assumptions, some owners may establish a minimum net present value prior to funding a project.

With the present values determined, a benefit-cost ratio can be computed. The benefit-cost ratio is simply the present value of total benefit divided by the present value of total cost. The larger the ratio, the more attractive is the project. In general, a benefit-cost ratio higher than 1.0 indicates that a project is economical, and the higher the benefit-cost ratio is, the more economical the project is. Conversely, with a benefit-cost ratio of less than 1.0, a project would be uneconomical, and with a benefit-cost ratio of close to 1.0, a project's economics would be marginal.

The benefit-cost ratio is very sensitive to the interest rate or discount rate used to calculate the annual costs of a project. If the interest rate changes, the benefit-cost ratio will also change. If the discount rate is increased, the benefit-cost ratio will decrease and vice-versa. Therefore, the selection of a correct or appropriate interest rate is crucial in calculating a reasonable benefit-cost ratio for the project.

If the annual benefits and costs of a project are not constant, or are not easily converted to a uniform annual value, then the benefit-cost ratio should be computed as the ratio of total discounted benefits to total discounted costs of a project.

However, it should be noted that in comparing two alternatives, one might have a higher benefit-cost ratio than the other, but a lower net present value.

2.7.3 Financial Considerations

Following the comparison of alternatives based on economic criteria, and assuming all other non-monetary requirements have been met, the final steps normally considered before moving forward to develop the project are financial. These relate to the company or utility's ability and willingness to allocate the required financial resources to the project. Changes in the business and regulatory environment are forcing a reduction in capital expenditures, requiring a new focus on efficiency and spending optimization.

Decisions on the preferred course of action and whether to move forward to implementation were previously driven primarily from an engineering perspective. Nowadays, the driver is the need to optimize asset performance and financial returns.

2.8 Moving Forward with a Life Extension or Upgrade Project

Several key points close this section about planning and evaluation, and they are all important.

- If additional information is required for informed decision-making, get it. Otherwise, be aware of the consequences of decision-making in the absence of all required information. Where there are gaps in the required information, fill the gaps with the facts. Consider what the risks are of making a decision without adequate information. Acquiring more information may be undesirable for some owners to pursue because it involves cost. And some owners are willing to accept the risk associated with incomplete information for a host of justifiable reasons.
- Don't accept project information blindly. Treat project information with care, and let it pass a "reality check" before it is accepted in the plan for service life extension or improvement.
- Consider using new technology to acquire necessary information. Chapter 3 provides further information on this topic.
- Check to see if the problem at hand has been solved previously. Case histories and industry contacts can yield a great deal of information that may shorten the distance to the solution and reduce the overall cost. Chapters 4, 5, 6 and 7 provide case history examples of solved problems.

2.9 Technical References

American Society of Civil Engineers (ASCE). (1997). *Guidelines for Retirement of Dams and Hydroelectric Facilities.* ASCE, New York, NY.

American Society of Civil Engineers (ASCE). (1992). *Guidelines for Rehabilitation of Civil Works of Hydroelectric Plants.* ASCE, New York, NY.

Electric Power Research Institute (EPRI). (1999). *Guidelines for Hydro Life Extension Modernization Guide: Volume I – Overall Process* [TR-112350-V1]. EPRI, Palo Alto, CA.

United States Army Corps of Engineers (USACE). (1984). *REMR Program (Repair, Evaluation, Maintenance and Rehabilitation Research Program.* USACE, Vicksburg, MS.

United States Bureau of Reclamation (USBR). (1980). *Safety Evaluation of Existing Dams.* USBR, Washington, DC. [A Water Resources Technical Publication.]

3.0 CHAPTER 3 - INNOVATIVE TECHNOLOGIES

3.1 Introduction

Chapters 1 and 2 describe the processes to extend the service life and upgrade hydroelectric civil works and outline the steps to better understand the issues, identify opportunities and recognize limitations. Chapters 1 and 2 also provide insight on gaining an understanding of the existing conditions and evaluating proposed changes that include identification and selecting the preferred alternative. Chapter 3 is a review of innovative technologies that have been used to extend, or upgrade the life of, many civil aspects of a hydroelectric project and has been written around seven activities (discussed in Table 3.1) associated with civil works. Subsequent Chapters; 4, 5 and 6, introduce the perspective of how problems with civil works features are identified, and the solutions evaluated and implemented.

Since the original construction of the dams, spillways and hydroelectric facilities which are now being considered for life extension and upgrades, there usually have been many developments in the technology. The drivers for the use of the new technologies for modernization of civil works are:

- The many structures and plants which are reaching the end of their economic lives and require decisions regarding their future, in a commercially focused environment.
- The increasing difficulty of constructing new dams and hydroelectric plants due to social, environmental and other regulatory concerns.
- The requirement for all water retaining structures to meet the necessary standards of current safety criteria.

The opportunities presented by the use of newer technology are that of lower cost and improved profitability, reliability, and environmental performance. These can be achieved through:

- Increasing output of the existing development.
- Reducing operating and maintenance costs.
- Increasing safety to acceptable levels.
- Reducing the effects of the facilities on the environment.

In general, the use and development of computer-based technology has been a catalyst for enabling many of these developments.

This chapter is primarily focused on innovative technologies for extending life and upgrading the civil components associated with hydroelectric facilities. Issues that are specific to dam safety assessment are covered in many other guides and documents, however this review identified many specific improvements developed in part to meet dam safety requirements that are relevant to all civil works structures. These include:

- Understanding the aging of dams and how to extend their lives.
- Understanding, detecting and preventing piping and internal erosion of embankment dams.
- Understanding and dealing with concrete dam cracking, Alkali-Aggregate Reaction (AAR) and frost damage.
- Understanding the behavior of materials (compacted earthfill in dams).
- Understanding the behavior of dams in extreme events such as earthquakes and floods.
- The increase in earthquake loadings and probable maximum flood levels and how to strengthen dams.

Technological advances that have been developed in response to these requirements include:

- Utilizing finite element analysis methods (FEA/FEM) in the evaluation of dams, and the understanding of dam behavior.
- An enhanced understanding of flood hydrology leading to more rational procedures for design flood estimation.
- An improved understanding of the nature and behavior of compacted earthfill in dams (including hydraulic fracture and the erosion characteristics of fill).
- The development and utilization of advanced ground engineering and grouting procedures.
- Improved understanding of seismicity, and of the seismic response of dams.
- Enhanced monitoring and surveillance capability achieved through improved and more reliable instrumentation and the development of associated assessment procedures.
- Improved infrared investigative methods (intrusive and non-intrusive).
- Development of AAR repair and monitoring techniques.
- Development of membranes/geotextiles for leakage and erosion control (internal and external).
- Development of performance based surveillance.

The information in this section is not a comprehensive guide to all new technologies, nor a checklist of items to be considered in any upgrade project. Rather, it includes a series of signposts inviting investigation into areas that may be of benefit to the project under consideration. These can also facilitate a watching brief on options in case changing circumstances should make one of them become attractive. These signposts cover:

- Technical developments.
- New materials.
- New methods of testing.

- New methods of analysis.

The innovative technologies in this section have been arranged under seven activities involved in life extension and upgrades of civil works and are shown on Table 3.1.

Table 3.1 Innovative Technologies For Life Extension And Upgrade

SECTION/ ACTIVITY	SOLUTIONS	CIVIL WORKS FEATURES
3.2 Surveillance Techniques	Underwater surveillance & remotely operated vehicles Automation of instrumentation & monitoring Flow measurement	Tunnels Intakes Dams Penstocks
3.3 Assessment Techniques	Condition assessment Business Risk assessment Testing	Spillways Gates Penstocks Hoists
3.4 Performance Analysis	Alkali Aggregate Reaction (AAR) Reduction of hydraulic losses Sealing Dams and Hydraulic Structures	Dams Spillways Tunnels Reservoirs Intakes
3.5 Life Extension Methods	Underwater repairs Construction methods Protective coating/relining & repairing water passages Repairing concrete & masonry dams Rehabilitation of gates Use of roller compacted concrete (RCC)	Tailrace Tunnels Intakes Penstocks Dams Gates
3.6 Modernization (Upgrades) Methods	Increasing operating heal	Flashboards Rubber dams
3.7 Environmental Considerations	Preserving historical structures Removal of hazardous materials Maintaining habitat Reservoir ice Reservoir debris Fish passage Reducing noise Water quality	Powerhouses Intakes Trashracks Fish Passage Spillways Tailraces
3.8 Management & Knowledge Systems	Management approaches Decision support Guides and standards	All components

(The references cited in the text are listed in full at the end of the chapter.)

Care should be taken, when assessing any of the new technologies, to review the risks of departing from tried and proven technology to the innovative. Some of the technologies described in the following sections do not, as yet, have a proven, long-term track record.

Manufacturers and suppliers of some of the products referred to in the following chapters may be found in Industry Sourcebooks, such as those published by Hydro Review (HCI Publications) and International Water Power and Dam Construction (IWPDC).

3.2 Surveillance

Technologies that relate to surveillance of civil structures for life extension and modernization include:

- Underwater surveillance and remote operated vehicles (ROV).
- Automation of instrumentation and monitoring.
- Flow measurement.

3.2.1 Underwater Surveillance and Remote Operated Vehicles

Remote Operated Vehicles (ROV), originally developed for the offshore oil and gas industry, provide a highly effective means of inspection and surveillance for hydroelectric projects, by reducing the high costs of divers, or for use in areas where dewatering, or use of divers, is not possible.

EPRI Hydro Technology Roundup On Remotely Operated Vehicles provides an excellent coverage of the present state-of-art, how the technology has developed, where it can be used and case histories of successful applications. The Technology Roundup also includes descriptions of various methods, their advantages and limitations, and their applicability to various situations. In addition, the report provides a framework for assessing the costs, benefits and opportunities for potential cost savings that might be obtained by using ROV technologies as part of an inspection and maintenance program (EPRI, 2002).

Other literature documents a number of case histories, which tend to cover inspections of closed systems such as pipelines and tunnels.

The requirements for annual inspection of a concrete lined tunnel led the owners to move from a costly dewatering exercise, firstly to divers, and then to inspections using ROV's. The examination of the concrete condition was videotaped and mapped using a computerized database. The case history includes a discussion of the inspection methods and results, and lessons learned, which include the suggestion to fully map and mark the tunnel lining during any dewatering (Hosko, 1995).

Another ROV system operator documented the benefits and justifications for their use, as well as three case histories covering power and tailrace tunnel applications. The benefits were not only the reduced costs associated with unnecessary dewatering and rewatering, but also the elimination of risks to tunnel stability in the dewatered state. The scope of the case histories includes (Clark, 2001):

- An unlined tunnel where it was suspected a partial collapse had occurred.
- A concrete by-pass tunnel where a valve and coupler failed and needed underwater repair.
- An unlined tailrace tunnel which cannot be dewatered and where higher than expected losses had occurred (Clark & Sherwood, 2000).

In another example ROV's were used in the inspection and surveillance of a reservoir from below the ice surface. This approach was selected to provide optimum water clarity to locate the source of leaks. The faults were located, video taped for permanent record, and their position documented. The equipment used, originally designed for pipeline work, river and ocean services and the inspection of hazardous sites, provides color images through a wide angled lens and uses a microprocessor driven magnetometer detection system on the surface (Fishers, 2003).

3.2.2 Automation of Instrumentation and Monitoring

There is a growing interest in the automation of instrumentation and monitoring systems, particularly for dam safety, but also for condition and performance measurements. The drivers are primarily improved data collection, but include the benefits of cost savings.

Sharing and utilizing automation technology improvements that have occurred over the last decade is the purpose of the *USSD General Guidelines for Automated Performance Monitoring of Dams.* This document focuses on the characteristics unique to automating instruments but does not address the fundamentals of monitoring and evaluating dams. As well as the technology, the Guide presents six case histories and has an extensive bibliography (USSD, 2002).

Increased surveillance measures were put in place following the discovery of two sinkholes on a very large dam. Monitoring improvements covered seepage, climate, water quality, displacement, video, and automated weirs and piezometers. A surveillance response plan and long-term monitoring plan were also put in place, based on a performance based surveillance program (a program structured to monitor the dam in relation to its hypothetical potential failure modes) (Scott & Hill, 2000).

Trends and technologies for monitoring dams include automation of instrumentation and early warning systems. The literature includes one example in which these systems were integrated, although it is emphasized that their use is site specific, and still requires the judgment of an experienced engineer (Nguyen, 2003).

Alkali-aggregate reaction (AAR) can cause seepage and cracking problems for a dam. Prior to remedial work, one dam was heavily instrumented and automated to provide data retrieval, as well as real time monitoring, during slot cutting while the dam was under load (Mochrie *et al*, 2001).

Site investigations and dynamic analysis showed that certain zones in the upper elevations of two earthfill dams could potentially liquefy under the maximum design earthquake. Remediation included lowering the reservoir level to remove and replace loose zones and upgrading instrumentation with remote monitoring. The monitoring included strong motion seismic instruments, weather station and reservoir levels, piezometers and weirs. All information and alarm signals from certain instruments are routed through a solar powered on-site control center, adequately protected from vandalism (Addo & Garner, 2002).

3.2.3 Flow Measurement

Monitoring the performance of individual components of hydroelectric plants is important for asset management and optimized operation. For two large diversion tunnels, which have never been dewatered, nor repaired, measurement of flows was required to more accurately define seasonal variations. A continuous and cost-effective flow measurement system using a dye injection method provided satisfactory results, though anomalies between results from the two tunnels still remain (Aydin & Burchat, 2003).

3.3 Assessment Techniques

The scope for innovative approaches for civil works in the area of assessment includes:

- Condition assessment.
- Risk assessment.
- Testing.

3.3.1 Condition Assessment

An approach developed to evaluate the condition of spillways uses a condition indexing approach. The basis is REMR, developed by USACE for the evaluation of numerous components of the power plant. This procedure has particular applicability for the analysis of complex spillway systems where operational aspects are critical (Chouinard *et al*, 2001). While developed specifically for the evaluation of gated spillways, it also can be used to prioritize maintenance activities (Chouinard *et al*, 2003).

Dam owners and regulators are placing increased emphasis on spillway gate assessment and evaluation to ensure their reliable operation when required. Experience and innovative procedures have resulted in improved results particularly where corrosion, damage or unrecorded modifications have reduced capability. This step-by-step approach assesses each component based on condition and performance (Edwards & Planck, 1999), (Edwards & Planck, 2001).

A scoring mechanism for the evaluation of spillway gates has been developed using an established software tool. The basic parameters used in the evaluation cover general condition, operating mechanism, condition of surface protection and life expectancy (Doujak *et al*, 2001).

One utility's experience with the assessment of flow control equipment, including both the mechanical and electrical components has resulted in the publication of three documents for use by the utility's own staff uses and any consultants hired to do this work. The standards cover for functional assessment of mechanical flow control equipment, and periodic reviews of mechanical and electrical equipment used in flow control. An important part of the work was the failure mode approach, in which components critical for equipment operation were identified (Barbour & Haines, 2002).

Citing the lack of information generally available, an assessment method for steel penstocks was developed (Ahlgren *et al,* 1995). This method involves five basic steps:

- Condition assessment, including information gathering and inspection.
- Data reduction.
- Analytical analysis to determine stress levels.
- Development of acceptance criteria, and.
- Structural evaluation comparing stress levels against criteria.

Assessment methods for condition and remaining life of penstocks and conduits are primarily based on determining that the corrosion protective coating and lining is intact and functioning (Stutsman, 1999). The process to assess condition and identify life extension needs includes:

- Data gathering and inspection.
- Thickness measurements.
- Operating head and pressure evaluation.
- Structural integrity assessment.
- Coating/lining rehabilitation.

An EPRI Hydropower Technology Roundup Report, *Steel Penstock Coating and Lining Rehabilitation* includes an assessment of current technology for evaluating a penstock's interior and exterior surfaces as well as guidelines for rehabilitation (EPRI, 1999).

3.3.2 Business Risk Assessment

Whereas condition assessment is primarily a deterministic approach to evaluation, risk assessment considers probability in either a quantitative or qualitative manner. The engineering assessment of risk and reliability has evolved from fields such as structural reliability, mechanical/electrical equipment and nuclear safety, where material properties and component failure are evaluated with statistical data. However, for the most part, risk assessment for the civil aspects of hydroelectric plants has developed much more slowly.

Penstock failure can cause numerous risks and consequences to a dam owner. Use of a risk assessment approach can identify and circumvent problems in a timely manner, increasing safety levels and reducing economic risk. A risk based life cycle management system was used to assess the risks associated with an uncontrolled flow through a ruptured penstock. Considerations of likely event, possible failure modes, likelihood of failure and consequences of failure were modeled to help in the decision making for remedial action (Westermann, 1999).

A major utility has developed a risk assessment methodology to assess penstock hazards and consequences to determine risks. The program is used to manage risk by prioritizing penstocks for evaluation, determining the cost effectiveness of various inspection programs and planning future work. The methodology involves estimation of failure rates due to hazards significant to each penstock, and the probable costs associated with possible failure scenarios. The data initially input to the risk assessment program is based upon historic failure information, drawings, specifications, start-up test data, etc. After the initial analysis, areas where mitigation measures, or additional information, would provide cost effective risk reduction, are identified and prioritized for future study. Results of the mitigation or studies are fed back into the analysis and the priorities are revised (Regan *et al*, 1995).

Effective maintenance of spillway gates and steel penstocks has been identified as a critical factor in the reliable operation of power plants. The move towards performance-based methods for maintenance systems includes reliability-centered maintenance (RCM) and life cycle cost assessment (LCC). Risk assessment of the structural assets is an integral part of the reliability analysis section of RCM (Maintenance work determination being the other section). Risk assessment is also an integral part of LCC where maintenance decisions are made from an optimum investment perspective (Yamamoto *et al*, 2003).

Risk assessment approaches to evaluating condition, performance or safety of civil components also highlight the relationship between upgrading and economic risk reduction. The US Department of Energy's Pacific Northwest National Laboratory (PNNL) spent several years modifying a probabilistic risk analysis methodology developed for the nuclear industry, to address specific hydroelectric situations. This included ways to analyze ways that plant operations might deal with abnormal events and accidents (Bardy & Gore, 2002).

This PNNL methodology was used to assess risks associated with existing gate closure systems, and possible alternatives for upgrade. Over nearly three decades powerhouse intake gates had been partially disabled or removed to prevent interference with fish bypass systems, or had been rendered inadequate or inoperable by lack of maintenance. The risk study results indicated that despite a number of uncertainties, the benefit of upgrading the intake gate operating system at any, or all, of the four powerhouses studied, is highly likely to exceed the cost by a high margin. Wire rope hoist systems or hydraulic cylinder systems were recommended. Either would allow emergency closure of the intake gates to comply with the owner's '10-minute rule'.

3.3.3 Testing

Testing is an important component in the assessment of civil structures.

To evaluate the condition and performance of spillway radial gates, a utility devised a stress and deflection measurement system. The results provided insights to the behavior of gate members, provided data to calibrate models for structural analysis, and determined the effects on trunnion friction where a new grease system was installed (Leach, 2003).

A cost effective testing and measuring system was undertaken for radial gate trunnion bearing friction. The results were used to analyze trunnion arm behavior and deflection and confirm serviceability and maintenance/rehabilitation requirements (MacTavish *et al*, 2003).

A practical and flexible instrumentation system was installed to collect data before and after refurbishment of radial gates. While the focus was on trunnion bearing friction, all components and systems that make up the entire gate were assessed. The testing took place under both loaded and unloaded conditions and some initial results are documented (Boyer *et al*, 2001).

During the operation of penstocks, fluid structure interaction can result in vibration response, although there are no moving parts. Fatigue damage is a function of loading cycles, and older facilities can become more prone to penstock failure. Operational testing of a vibrating penstock can help determine whether structural degradation is occurring. Analysis can be used to determine remaining service life (Todd, 2003).

To define the need and timing for the installation of protective lining in a 14-year old conveyance system, a large utility employed an extensive inspection and monitoring program combining ultrasonic, visual and electrochemical components. Previous semi-quantitative in-service inspections had provided data for buckling analyses and to determine timing of lining installation if deemed necessary. In the new 3-step program, an automated ultrasonic thickness measurement system was used to determine existing thickness, visual inspections characterized the common product on all inside surfaces and a corrosion monitoring system was installed to define future wall loss (Mattson & Ahlgren, 1999).

3.4 Performance Analysis

Performance issues that have led to the development of innovative approaches to modernization and life extension include:

- Alkali-aggregate reaction (AAR).
- Reduction in hydraulic losses.
- Sealing of dams and hydraulic structures.

3.4.1 Alkali-Aggregate Reaction (AAR)

AAR is a contributing factor for concrete growth, deterioration and distress. It is a chemical reaction in concrete that occurs when alkali in the cement reacts with certain silicate or carbonate minerals in the aggregate. This chemical reaction results in a gel that absorbs moisture and swells (Wagner & Meisenheimer, 1997).

Concrete structures experiencing AAR generally demonstrate some of the following characteristics:

- Spillway and other openings may show closure or tilting, resulting in binding of spillway, intake and lock gates.
- Dams may experience tension cracking due to the differential growth rate between the downstream face and the interior of the dam.
- Generating equipment structures may experience ovalling due to compression between units affecting clearances and attachments.
- Plumbline readings for a dam may indicate an upstream movement.
- Expansion and contraction joints can permanently close.

Before a management plan can be developed, data regarding the effect of AAR on the dam in question needs to be gathered. It generally takes at least three annual seasons before trends from AAR growth can be separated from annual thermal changes in massive concrete structures.

Some general guidelines have been prepared to assist in recognition, with discussion on testing, analysis and remedial works that have been found appropriate. However, it is stressed that in addition to an initial focus on detail and problems, it is important to have a broad understanding on overall structural integrity (Mason, 1998).

As part of the initial management process of AAR affected structures, a classification system of mass concrete swelling mechanisms can be adopted. One system classifying the swelling mechanism into 4 types; chemical, hydraulic, mechanical and hydroelectric thermal creep has been proposed (Lupien, 1995). Understanding the cause allows appropriate remedial actions to be selected.

An extensive review of international experience and practice and the progress made in long-term management of AAR-affected dams and hydroelectric projects includes over 100 structures. From this database, and some case histories, an analysis has been made of issues affecting dam safety, power plant operation, remedial measures and the long-term performance of AAR-affected structures (Charlwood & Solymar, 1995).

A specific management plan has been developed for a large hydroelectric plant built in the 1960's. Since the 1980's the management approach has been monitoring, analysis and design of remedial works. These include the use of slot cutting to control deformation, tendons in the intake, a grouting program and in innovative coupling installed in the penstocks. Ongoing monitoring and necessary remedial works will significantly extend the life of the plant (Gilks *et al*, 2001).

A program for long-term management of AAR-affected dams and hydroelectric plants has been developed based on a framework of performance parameters. These relate to the identification of potential failure modes, key performance indicators, instrumentation and monitoring systems and remedial measures. Using such a formalized approach can be useful for the monitoring and identification of AAR-affected structures (Charlwood, 1995).

Finite Element Modeling (FEM) has allowed cost-effective development of detailed models that can simulate the behavior and performance of AAR in structures. The models can also be used to evaluate alternatives that can be used to mitigate the effect of AAR (Wagner & Meisenheimer, 1997).

The repair techniques for AAR include:

- The use of stress relief slots by slot cutting (diamond wire saws) to allow room for the concrete structure to 'grow'.
- Gate repairs including adjustments to gate tolerances as well as flexible gates or slots.
- Shear control by the use of post-tensioned anchors.

FEM can be used in conjunction with these repair techniques to predict requirements for future slot cutting maintenance, additional slot locations and estimated time requirements for installation and loading increases in anchor systems.

Remediation of AAR-affected structures is well documented and a few case histories, which indicate the breadth of the challenge and innovative ways adopted, are presented. Deep slot cutting is an appropriate and proven method to manage deformations and excessive stress concentration due to AAR and thermal effects and of improving structural behavior of concrete dams. The aim of this technology is to relieve internal stress and to create an effective expansion joint which can accommodate reversible and irreversible displacement induced by thermal cycles as well as permanent movement due to chemical concrete swelling caused by AAR.

Slot cutting needs to be analyzed and designed carefully as it is a major intervention that affects structural integrity. Cutting a number of slots at intervals and strengthening the structure with post-tensioned cables is preferred. Instrumentation of the slots to monitor long-term behavior is part of the program (Veilleux, 1995).

Innovative technology used for slot cutting includes methods to seal the cut and drilling technology (Szita *et al*, 1995).

A large power plant has experienced many operational and structural problems, including decreased generator air gaps and turbine runner clearances, commencing in the 1970's. In addition, the powerhouse concrete structure was deteriorating with extensive cracking and water leakage. Concrete expansion due to AAR was diagnosed as the root cause of concrete movement and ensuing generator and structural problems. A proactive concrete rehabilitation program was implemented to mitigate the effects of AAR induced concrete expansion and repair the concrete structural damage. Slots were successfully cut between the generators along the expansion/contraction joints of the concrete structure, using diamond wire technology. The immediate results were encouraging with reduced compressive stresses in the concrete, increased runner clearances and partial rounding of the throat ring liners. The slots were also sized to provide allowance for future concrete expansion. Numerous innovative techniques were implemented to rehabilitate the structural components that were severely damaged by concrete expansion at the powerhouse, including specialized grouting and sealing technology (Eastman *et al*, 2001).

Radial type spillway gates at a large dam have a history of binding problems caused by growth of the concrete piers and non-overflow sections. The concrete growth has resulted in distortion of the embedded side seal channels. The two end piers have been subjected to the largest amount of concrete growth, reducing the width of the two end gate bays permanently. The sides of the gates have had to be shaved several times so that the gates could be operated without binding. Further shaving is not possible without compromising the structural integrity of the gates (Sehgal *et al*, 1995). Modifications include:

- Replacing the gate side embedded parts including cutting new radial slots in the piers.
- Reducing the gate width so that future growth will not interfere.
- Modifying the gate side and bottom seals.
- Modifying the gate trunnions.

A concrete gravity and a concrete buttress dam experienced the development of cracking and deformation, found to be caused by AAR. As part of the rehabilitation, a waterproofing PVC based geocomposite was installed to eliminate water intrusion and to protect the facing from further deterioration. The installation system allows drainage of the infiltrated water, thus allowing dehydration of the dam body. On one dam, the membrane also provided protection for future slot cutting (Scuero, 1995).

3.4.2 Reduction of Hydraulic Losses

An effective means of improving the output and value of hydroelectric plants is to reduce losses through the hydraulic systems. For projects which have long tunnels, hydraulic losses can be significant and performance improvements can be cost effective. This is particularly the case where the long pressure tunnels were constructed by the drill and blast method. Many of these older unlined tunnels were designed and constructed considering the economics of the time and the expected operation of the power plant at the end of the power tunnel/penstock. Depending on their design, these longer tunnels may experience high relative frictional losses if operated at or above their original design capacity. If the capacity of the generating units is increased, hydraulic losses will increase significantly. Some conventional methods to decrease the friction in existing unlined tunnels have been to either concrete line the tunnel, pave the invert, or remove rock projections.

Drilled and blasted unlined rock tunnels in Sweden have been analyzed for their macro roughness characteristics in a series of studies. Field measurements and laboratory investigations provide basic insight into the hydraulics of such tunnels when flowing full. The macro roughness of drilled and blasted tunnels depends primarily on the variation of sectional area. The tunnel roughness can be significantly reduced by smoothening of tunnel walls for example, shotcrete lining. Planning and design of tunnel uprating procedures to reduce head loss must be based on methods for estimating the roughness both before and after the tunnel rehabilitation. From this the economic values of head loss and additional energy production, the investment cost of lining and the optimum tunnel are determined (Elfman & Cederwall, 1999).

An innovative approach to reduce frictional losses in an unlined tunnel at a low cost was using a hammer mounted on a small excavator to selectively remove rock projections in the tunnel flow streams. This project had a short payback due both to improvements in the flow regime and the limitation of maximum capacity during peak plant loading (Gass, 2003).

There are also opportunities to reduce hydraulic losses where a tunnel is lined. A case history is described where the intake concrete tunnel at a large hydroelectric plant is coated with a black sticky substance (slime) approximately 5 mm in thickness. Since the tunnel is 12 km long, this deposit represents a considerable obstacle to the production of electric power and the resulting loss of power generated is significant. Inspections showed that simply cleaning the surfaces reduced the power losses but the slime built up again after a few years. A thin, smooth protective coating, containing anti-slime agents, compatible with the existing concrete could protect the surfaces against erosion and limit the slime deposit and its harmful effect on power production. A protective coating using a polymer-modified cement-based mortar, was applied to a small surface area of the tunnel during a generating station shutdown. The size of the tunnel, its restricted accessibility, cleaning, ecological disposal of the slime, and the large quantities of material to be applied to cover the entire tunnel added to the complexity of the project (Mirza *et al*, 2001).

Reducing hydraulic losses along an existing tunnel is one way to improve performance. Other ways, though much more expensive have been adopted, including increasing the cross sectional area of the tunnel by further excavation or construction of a new adjacent tunnel (twinning). While the first method requires an extensive period out of service, the latter can be accomplished for the most part without interrupting generation.

Section 3.5 includes examples where water passages (conduits and penstocks) have been repaired and relined to extend life. This process can have the additional benefit of reducing hydraulic losses.

Due to severe corrosion damage, a decision was made to reline a penstock to allow continued operation. If this work had not been carried out, complete replacement of the penstock would have been required. However, one of the most important benefits was the reduction in head loss. Index tests carried out before and after the work showed a system head loss reduction of 39% which translated into increased generation potential of 3.3% or 3.2 GWH/yr (Stutsman, 1997).

3.4.3 Sealing Dams and Hydraulic Structures

Dam leakage mostly occurs through cracked or deteriorated concrete and defective joints. Conventional repair techniques generally consist of localized sealing of cracks and defective joints by cement and chemical grouting, epoxy injection, or surface treatments.

Geomembranes are also used for dam repairs. They have been used as synthetic barriers in dams for more than 30 years. In recent years, they have also been used for seepage control and have been used to successfully resurface the upstream face of old concrete and masonry dams, particularly in Europe. Mostly, however, the installation of geomembranes has taken place in dry situations requiring dewatering which can be expensive and not always possible with project constraints. A technique has now been developed for the installation of geomembranes for underwater repairs (McDonald *et al*, 1997).

A multi-layer sealing system based on PUR ductile plastic successfully addresses long-term dam sealing goals and requirements. Earlier PVC geomembranes had such problems as difficulty with rock anchorage, vulnerability to damage by sun, ice and vandalism, and underseepage when damaged. In this new system, each layer has different characteristics and the layers are integrally bound during setting, with each forming a sealing membrane. It is suitable for various substrates such as concrete, rock or steel, and can be used on very rough substrates such as natural rock face, providing the surfaces are re-profiled first by spraying with shotcrete (Lardi *et al*, 2002), (Rűesch & Lardi, 2003). Additional features include:

- Suitability for partial sealing situations, e.g. dam crests, rock faces.
- Prevention of alkali-aggregate reaction.

- Suitability for concrete/masonry dams, reservoirs, intake/outlet structures and canals.
- Joint detailing is possible without any mechanical fixings or templates, tapes or textile inserts.
- High tear stretch and strength.

A drained geomembrane was selected to seal the upstream face of a masonry dam affected by uplift pressure and seepage after previous rehabilitation measures using cement grouting and a later shotcrete lining had proved unsuccessful. The geomembrane was used in conjunction with an internal drainage and control gallery, and the installation of drainage holes with piezometers from the gallery to the bedrock. The geomembrane is a geo-composite flexible polyvinyl chloride. The UV resistance provided by titan-dioxide additives allows for installation without a protective layer. Installation of the geomembrane seal on the upstream face and drainage of the bedrock involved (Schewe, 1999):

- Removal of the upstream clayey sealing fill.
- Cleaning, removal of loose parts and rehabilitation of the damaged and deteriorated shotcrete slabs.
- Placement of a geomembrane on top of the shotcrete slabs.
- Installation of a concrete toe beam at the upstream dam toe.
- Excavation of a control and drainage gallery in the masonry at a low level.
- Installation of drainage holes with piezometers from the gallery into the bedrock.

Increasing leakage at an old concrete faced rock fill dam necessitated repairs and a number of alternatives were considered. These included a concrete face repair, face patching and joint repair or resurfacing the entire dam with concrete, asphalt or a synthetic liner. Repair to the vertical joints was selected, however the concrete adjacent to the joints had excessive freeze-thaw damage. The damaged concrete was removed, the joints refilled with shotcrete and an exposed geomembrane system was installed over the new concrete surface (Larson, 2003).

The underwater installation of a geomembrane at an arch dam was planned to coincide with the rewinding of a generator to allow significant drawdown of the reservoir without loss of power generation, reduction of dive depths, and hence increased underwater dive time (Scuero *et al*, 2000). The water proofing system is a composite PVC geomembrane heat coupled during fabrication to a polyester geotextile. To address the underwater work, and to optimize the construction schedule, standard sheets were pre-assembled into larger panels. Custom-made panels avoided the need for transverse joints. Other features included:

- Stainless steel profiles which provided watertight connections between panels, tension to avoid slack and acted as vertical pipes to drain water.
- Stainless steel batten strips to secure PVC liner to dam face and provided a watertight seal along entire periphery.

- Drainage system behind the PVC geocomposite designed to intercept water seeping from the foundation, rain and snowmelt and condensation water migrating to warmer upstream face.
- Monitoring system designed to investigate and control the discharged water and measure water levels behind the geomembrane from the bridge deck.

Three examples of successful synthetic geomembrane use in the rehabilitation of existing dams, show the versatility of their application. A 25m high concrete dam with a partial masonry facing, subject to extreme climate and freeze/thaw conditions, as well as alkali aggregate reaction, was fitted with a composite membrane of PVC and geotextile. At a 40m high embankment dam with an upstream bituminous concrete layer, a PVC geocomposite liner was also used, in this case directly over the deteriorated surface. At a 36m high arch dam, subject to spalling and loss of concrete, part of the construction of the PVC composite was conducted under water (Anon, 2000).

3.5 Life Extension Methods

The repair of aging dams and hydraulic structures is an important consideration for the life extension and modernization of hydroelectric facilities. Some typical problems, which have prompted the search for innovative repair techniques include:

- Underwater repairs.
- Construction methods.
- Relining and repairing water passages.
- Repairing concrete and masonry dams.
- Rehabilitation of gates..
- Use of roller compacted concrete (RCC).

3.5.1 Underwater Repairs

Tailrace piers are prone to distress from the effects of erosion, abrasion and freeze-thaw mechanisms in cold climates. Traditionally, conventional concrete repair techniques have been employed for this sort of deterioration, usually involving the installation of a cofferdam to enable repairs under dry conditions. Recently new methods have been developed for underwater repair of concrete. One such method is Advanced Polymer Encapsulation (APE) which uses a fiberglass reinforced jacket and polymer grout (FRP) (Deans *et al*, 2003). The system has been found by one major operator to have the following advantages:

- Significant reduction in unit outage times.
- Significant technical and environmental advantages.
- Energy cost savings due to replacement energy cost difference.

3.5.2 Construction Methods

The provision of a temporary bulkhead within the existing tailrace, at a point close to an underground power station, was the method used to create a dewatered area in which to accomplish the junction between existing and new tailrace tunnels. The bulkhead solution provided several major advantages over the original proposal to build a cofferdam in the outlet channel, and significantly reduced the dewatering necessary to undertake the tasks. The total outage time to complete the connection of the two tunnels was halved to eleven days (Caufield, 2003).

In replacing the deteriorated spillway deck and hoists at a small hydroelectric plant, an owner faced numerous constraints related to time, economics and river flows. Comprehensive project planning and innovative techniques for demolition and construction were used to complete the project successfully. Removal of the old deck and piers was the most challenging aspect and the method used was a diamond cutting wire saw. This allowed removal in one piece, saved time and avoided material falling into the river (Rudolph & Campbell, 2003).

Many hydroelectric plants are between 70 and 100 years old and many structures have experienced significant deterioration of concrete steel support elements. Rehabilitation often involves the removal and replacement of deteriorated materials and temporary support or underpinning is required. The size and weight of these massive structures provides a significant challenge. Three case histories are documented (Jones *et al*, 2002), where underpinning was used:

- The deteriorated downstream face of a multiple arch concrete substructure of a powerhouse.
- The concrete rock lining supporting a pre-stressed concrete surge tank which had suffered extensive freeze/thaw damage.
- The cracked section of a powerhouse wall.

An artificial ice plug developed in Norway makes it possible to close a water-filled tunnel, where there are no upstream gates, or other easy ways to cut off the water, so that rehabilitation can be carried out. The concept of the ice plug is based on the principle that ice, soil and rock acquire greater strength when frozen, and that the ice structure can be removed simply by melting. It is thus ideal for providing temporary protective structures, such as are required during rehabilitation work (Berggren & Sandvold, 1995).

3.5.3 Relining and Repairing Water Passages

The rehabilitation of deteriorated reinforced concrete water retaining structures between the intakes and steel penstocks of an 8 unit 432 MW plant, provided permanent solutions to problems particularly associated with aging hydroelectric facilities. The transition structures had leaked since commissioning. Poorly bonded concrete construction joints were the main sources of water seepage together with cracks that developed with time. The saturated state of the concrete around the joints and cracks, the lack of entrained air in the original concrete and exposure to severe winter conditions resulted in extensive freeze/thaw deterioration of the concrete.

The solution to the transition leakage problem was to install steel linings in the transitions from the headgate roller paths to the upstream ends of the steel penstocks. New stainless steel roller paths were also installed in the existing headgate gains, together with new sills and lintels.

A state-of-art expansion joint system intercepts water seeping along the outside of the penstock pipe and drains it to the powerhouse roof drains located around the penstock envelope. A pressure-grouting system using four different types of grout was implemented to eliminate any potential water seepage paths affecting the penstock transition liners and penstock pipes (Carrington *et al*, 2002).

Advanced composite technology has expanded the options for life extension and modernization methods. Advantages of composites include:

- No hot work.
- Suitability for confined access.
- Easily handled.
- No pre-fabrication.
- Multi-axial strength up to 10 times that of steel and twice as stiff.

A carbon fibre/epoxy resin composite was applied to severely eroded and corroded 15 inch (30cm) ductile iron scour pipe using a wrapping technique. The material enabled restoration of structural strength and pressure integrity without increasing the size of weight of the existing pipelines. An innovative clamp was also engineered to strengthen the intersection of the composite wrap and the rock. Tripod legs resting on the tunnel floor are integral to the clamp support, and this feature, combined with the extension of the composite over the interface, ensures pressure containment and overall structural integrity (Anon., 2002).

An innovative remedial plywood lining has extended the life of a 46-year-old wood stave penstock. Previous measures to address leakage using new wood inserts with steel plate covers, or HDPE plastic sheets, had been reasonably successful, but a more permanent solution was sought to allow additional life of some five years before a major plant redevelopment. An internal plywood liner offered a number of advantages:

- Allowed small amount of leakage to prevent dry rotting of the wood stave and stave shrinkage.
- Simple, low-tech solution.
- Compatible with existing material.
- Required no special tools.
- Required minimal disassembly of existing penstock.

- Materials easily handled.
- Minimal outage time.
- Cost-effective.

Waterproof marine-grade plywood was dry-bent in place and secured with screws. Sheet joints were staggered and caulked with a monomer sealant. This technique has merit at other projects where deferring the capital cost of penstock replacement may justify an economical interim repair solution (Clarida, 1999).

A wide range of alternatives is available for upgrading water conveyance systems nearing or reaching the end of their useful life. One utility has implemented two different innovative and cost-effective methods. In one case wood-stave pipe has been replaced with 10-foot diameter HDPE pipe, considered to be the most economic solution. The pipe is relatively easy to move with standard hydraulic equipment and can be readily cut in the field using a small chain saw. Concrete was used to construct transition structures where the HDPE pipe interfaced with the existing conveyance system.

At another of its sites, PVC liner was installed in a wood-stave flow-line pipe where excessive leakage was causing substantial losses to power generation, and damage had been caused by a shutdown. Flexible polyester reinforced PVC sheeting, (pond liner) was chosen for use with aluminum hoops at 10-foot intervals. The result was a 90% reduction in leakage, and reduced ground surface erosion. However, when a combination of high velocities and low head pressure subsequently caused damage to an upper section of the liner, the design was modified to reduce liner sag in a second installation of the 12-foot diameter wood stave pipe. Additional aluminum flat bars were placed longitudinally to the flow area (Atwood *et al*, 2003).

Faced with leaks in its siphon to a small hydroelectric plant, an owner opted for a trenchless technology using a cement mortar lining called Spincrete. This approach, which applies the cement mortar lining centrifugally, avoided the expense of replacing an extensive length of pipe. The owner expects no maintenance will be required for 20 years on the treated sections, with life extension up to 70 years on certain projects. However, the process has limitations for repairing unsound structures (Heinrich, 2003).

A remedial project replaced the linings of deep sluices seriously eroded by a high sediment load. The sluices, which serve to maintain flow through the reservoir at certain times of the year, were originally lined with 3mm stainless steel, but had eroded principally as a result of their age (30 years). Of a number of possible solutions, a martensitic stainless steel with a hardness of 450 HB was identified as the material sufficiently abrasion-resistant to provide a long design life, to the next major refurbishment (Denny, 1998).

A synthetic aggregate for abrasion-resistant concrete has been developed to withstand heavy sediment loads, and high velocities of 35m/s or more. Slag obtained as a by-product of copper production is transformed into a high quality, entirely crystalline and extremely tough material that can be designated as olivine-spinel rock. Concrete mixed with this synthetic aggregate has been shown to be twice as resistant to abrasion as that mixed with natural aggregates. Operators of an Austrian run-of-river plant where it has been used considered that future savings in repair and maintenance costs and generation losses will outweigh increased initial costs (Anon., 1998).

3.5.4 Protective Coatings

The application of the appropriate coating is an important consideration for the protection of civil structures associated with hydropower plants. Painting or other coatings will normally have been part of the original installation. After many years of operation, decisions often have to be made on rehabilitation or replacement. Recoating of structures, if done in a timely manner, will likely be much less expensive than a full or partial replacement, and can provide many additional years of operational capability. Some examples of innovative coating applications for penstocks (inside and outside) are gates are summarized as well as a reference to foul release coatings tests.

A penstock rehabilitation project included steel surface preparation and application of a protective coating selected for the site specific conditions. It was initially planned to sand blast and remove the existing coating, which contained high levels of lead. The selection of the new protective coating allowed recoating, after pressure washing and was completed for a fraction of the cost (Dobrowolski, 1999).

A steel pipeline/penstock had large areas of the original interior tar coating damaged by rust. The selection of the appropriate new protective coating was critical, especially as work was to take place in the winter and there was a fixed schedule duration. Access to the approximate 1.2 mile (2km) pipeline was through a mid-point cut out, which allowed work in both directions. Old coating removal and application of the new, thick-layer protective coating was completed within 90 days (Zwanzinger, 2003).

Gate rehabilitation at existing hydropower plants are usually driven by safety concerns and/or risks associated with aging. The risks are assessed by consideration of condition, probability of failure or malfunction and the ensuing consequences. In the situation where replacement gates will be very inaccessible, the selection of the paint system was particularly important. An epoxy mastic coating was selected and careful attention was placed on inspecting the application (Kahl, 2003).
Eighteen non-toxic foul release coatings were tested on two power plants which had experienced zebra mussel infestation. Each coating was compared based on three primary characteristics; effectiveness in reducing the level of bio fouling, economy in application, maintainability and durability, and environmental acceptability in not releasing contaminants. The tests provided a wide range in results (Wells, 1999).

3.5.5 Repairing Concrete and Masonry Dams

Using commonly available construction materials in unconventional applications provides cost-effective and reliable solutions to many typical "aging" problems. Sealing stop log stacks by applying geomembrane materials *in situ,* has been shown to reduce leakage by 95%, and substantially reduce icing problems. The injection of polyurethane foam into ice boom logs increased buoyancy by 50-60%. PVC sheet pile materials provide non-conductive, non-corrosive, and light resistant walls for oil containment pits. The relatively low cost material can be easily handled and transported in close proximity to operating transformers (Clarida *et al*, 1999).

A twelve-year research project has led to an effective and durable solution to "plugging" cracks in one of its concrete arch dams: a new grout injection campaign. Outcomes of the project highlight the importance of understanding the rheological, physical and mechanical properties of different grout media for specific environmental and structural conditions (Saleh, *et al*, 1999). The performance and applicability of various grout types were assessed in relation to:

- Size of cracks.
- Site temperature conditions and variations.
- Compatibility between grout and admixture.
- Fresh and hardened behavior.
- Optimum composition of the mixture.

In this case, water temperatures of 4-5° C dictated the use of a cement-based product.

In addition to accomplishing a safe and durable injection, the subsequent grouting campaign aimed to:

- Control the injection to ensure effective grout penetration and avoid putting undesirable pressure on any other undetected cracks.
- Eliminate water infiltration through the cracks and reduce the water pressure within the structure.
- Increase the shear resistance of the structure at the level of the plunging cracks.

Data collected prior to, during and after the injection campaign has provided broader benefits:

- A better understanding of the dam's structural behavior before, during and after injection.
- Improved knowledge of the development and response of plunging cracks.
- Much new information about grouting materials, equipment and methods.

An owner of a large inventory of concrete dams has an ongoing program of refurbishment and modernization. Driven mainly by the ageing of the concrete (deterioration), and changes in technology and criteria used, this has resulted both in increased operational and maintenance issues, and dam safety concerns. A case history of a modernization project is described where a highly fractured zone of concrete was identified and a mass concrete buttress was used to provide stabilization. A summary of eleven other refurbishments is also documented (LaBoon, 2002).

3.5.6 Rehabilitation of Gates

As part of a civil rehabilitation program, a utility sought a cost-effective way to dewater its existing spillways, intakes and draft tubes. More than eighty water passages had been constructed with upstream stoplog guides but without dedicated gates. Working with consultants, the utility devised an innovative and economical solution: two types of expandable steel gates that are not site-specific, but can be adapted to dewater water passages of varying size. The 'telescoping gate' is used to dewater small structures with openings of 8 to 20 feet (6m to 16.8m). The components for this gate come in various sizes to span openings of various widths. Both types of gates have been successfully employed at multiple sites. Neither type requires special storage (Westermann, 1998).

Many small hydroelectric plants were constructed 50 to 100 years ago. These plants have reached a point where structural repairs to spillway and other gates are becoming commonplace and some of this repair work involves fastening steel members and components to concrete superstructures in a submerged condition. If it is not possible or economical to dewater the area in question, an adhesive anchor with an acceptable delivery system can be utilized to achieve the desired results. A case history is described where a spillway bay was dewatered in order to replace four original crest gates. The dewatering cofferdam consisted of structural steel stoplogs fitted into bearing plates, which were attached to the concrete abutments. The method of installation set approximately eight hundred adhesive anchors in horizontal holes drilled in concrete abutments, in approximately 10 to 18 feet of water (Riddle, 1995).

Rehabilitating gates on old dams can be a challenge. In planning for a cost-effective rehabilitation, the benefits of repairing versus replacing the aged concrete and steel components need to be carefully evaluated. A gated spillway, constructed in the early 1900's has 13 Stauwerke self-operating flap gates. Each gate uses three pivotal trunnion beams and internal concrete counterweights to enable the gate to automatically open progressively as reservoir levels surcharge above normal pool. Similar in operation to a leaf gate, the Stauwerke gate rotates seals across side plates in a downstream to upstream motion (in contrast to the operation of a radial gate) (Rudolph & Gundry, 1995).

A rehabilitation plan was adopted to address the following critical items:

- Verify that the gates would fully open to pass design flows.
- Repair or replace electrical, structural and concrete components as required.

- Remove or encapsulate lead based paint and paint components to extend life at least 15 years.
- Minimize icing problems by reducing gate leakage and using weatherizing techniques.

3.5.7 Use of Roller Compacted Concrete (RCC)

RCC is more a new construction method than a new material. It is a true concrete that is usually mixed in a pug-mill mixer, transported by dump trucks, large front-end loaders or conveyor belts, spread by dozers, and compacted by a vibratory roller.

In comparing RCC with conventional slump concrete, less water is needed to achieve a no-slump consistency. Therefore, less cementitious material is required to produce an equivalent water/cement ratio. Less water in the mixture leads to less shrinkage and less cement is one means for reducing thermally induced cracking.

RCC has become one of the most popular construction methods for upgrading both embankment and concrete dams. The main applications for RCC to rehabilitate existing dams have been its use to provide overtopping protection for embankment dams and to strengthen concrete dams by the addition of a downstream buttress (USSD, 2002).

The widespread acceptance of RCC for increasing the hydraulic capacity or structural stability of older dams is based on its low cost and rapid construction. Rapid construction leads to low cost and less cement and the possible use of less costly aggregate in the RCC mixture. This, together with little or no forming and usually no reinforcement, leads to a lower unit cost for the concrete. Quality is another reason for its frequent use in dam rehabilitation projects, as RCC has excellent abrasion/erosion properties when exposed to flowing water and can be produced to any desired compressive strength to meet project structural and hydraulic requirements. A review of RCC dam rehabilitation projects in service has found them to be quite durable, requiring little to no maintenance.

Other added benefits of these rehabilitation methods are related to construction on the downstream side of the dam. In many situations, there may be no need to lower the reservoir in order to accomplish the upgrade and environmental effects of the construction may be minimal, limited to the dam and downstream toe areas.

A major rehabilitation scheme included constructing a new emergency spillway and replacing the existing primary spillway. Special challenges have included a tight schedule, extreme climatic conditions, and the need to keep one of the two spillways operational throughout. RCC was selected as a primary construction material, taking advantage of locally available aggregates; this proved to be a cost-effective solution, which also saved construction time. An important criterion of the rehabilitation plan was to minimize the head necessary to pass design floods. A labyrinth weir crest was selected for the primary spillway and a stepped chute emergency spillway was located on the earth dam. RCC was used for the emergency spillway (Yadon *et al*, 1998).

To meet the need to increase a spillway discharge capacity and enhance stability, the dam's owner chose to build a new spillway, constructed mainly of roller-compacted concrete (RCC). Five alternatives were evaluated including conventional concrete, anchoring an inflatable rubber dam and RCC designs. A new structure in both conventional concrete and RCC was tendered, with RCC selected based as lower cost and shorter construction schedule (Locke & O'Neil, 1999).

3.6 Modernization (Upgrade)

Modernization of civil works is dependent on the economic benefits and the regulatory constraints that affect them. Where feasible and allowable, modernization of civil works can take advantage of many innovative developments, mainly related to increasing operating head.

Many older dams and spillways controlled water levels with stop logs, flashboards and manually operated gates. An increasingly popular way of increasing the reliability and safety of such devices, as well as improving economics, is modernizing these spillway control systems.

There are many case histories relating to the use of inflatable rubber dams for both new structures as well as existing ones. In one such case, an existing spillway structure with stop logs needed refurbishment and an increased capacity. Following removal of part of the structure an inflatable rubber dam of 17 feet (5m) diameter was installed (Sonier *et al*, 2001).

Operations, economic and safety considerations were the justification for replacing wooden flashboards with an inflatable rubber dam. The flashboards were being repeatedly damaged or lost due to flooding and debris, resulting in power loss and requiring manual replacement. The inflatable rubber dam was designed to withstand both the debris loading and very cold weather conditions (Kmetz *et al*, 1995).

The use of fusegate systems is another innovative method to improve spillway control. A proposal to raise the operating water level of a dam by over 6 m considered a variety of options. The selected fusegate system forms a watertight barrier at the existing spillway sill. All floods up to a certain pre-determined magnitude will pass over the fusegates, after which they will tip over (Bowers *et al*, 2000).

A hybrid system using a fixed reinforced concrete labyrinth system on the main spillway and a fusegate system on the auxiliary spillway allowed a dam to be raised 2 m. This combination significantly improved safety levels, as well as significantly increasing storage volumes in the reservoir at a modest cost (Shaw *et al*, 1999).

A double failure of rubber dams provided a search for a more reliable and safe method for reinstating reservoir storage capacity. Citing their inherent reliability and minimal maintenance requirements, the operator selected a fusegate system (Dohnálek *et al*, 2002).

The need to replace aging gate equipment, together with the evolution of new safety requirements, led to the selection of a fusegate system by another operator. Model testing was used to confirm design parameters for the tallest (5m) fusegates installed in Europe to that time (Comte *et al*, 2000).

While ungated spillways offer the safest performance, they are normally less economical due to the volume of water stored. Development of innovative water control systems that are fully automatic have over 20 years of operating history in South Africa. From this experience, a new generation of spillway gates has been developed. Of the self-actuating systems, there are two main types, fuseable and restoring. The fuseable ones (such as the fusegate) open automatically, but need to be replaced. The restoring ones open and close automatically, based on an actuating device (Townshend, 2000).

3.7 Environmental Considerations

Environmental issues often affect decisions around life extension and modernization works. Innovative approaches that have been developed to address these issues include:

- Preserving historical structures.
- Removal of hazardous materials.
- Habitat maintenance.
- Reservoir ice.
- Reservoir debris.
- Water quality.
- Fish passage.
- Noise reduction.

3.7.1 Preserving Historical Structures

Hydroelectric plants, their powerhouses, dams, reservoir infrastructure and ancillary works may have historical significance because of their age, location, design, architecture or their overall contribution to hydropower development.

These plants may already have a historical designation or are eligible for listing as a Historic Place. However, as they age, are no longer considered safe, require upgrading or have to be re-licensed, some aspects of historical significance could be lost. A set of guidelines for preserving the historical significance of hydroelectric plants is specifically related to the preparation of a Historic Properties Management Plan. Produced by FERC and the Advisory Council of Historic Preservation, these guidelines will assist owners and licensees in preparing a plan that can be readily integrated into the plant modernization process (Campbell & Dean, 2003).

3.7.2 Removal of Hazardous Materials

For powerplants built in the 1930's, asbestos materials were commonly used as an insulator.

During a recent modernization project, these materials, now considered hazardous, required removal prior to the major work activities. To ensure no possibility that asbestos particles would become airborne, a full abatement program was initiated, including containment, isolation, proper handling and safe disposal (Alberty, 1999).

A penstock rehabilitation project, described in 3.5.4, covered the encapsulation of lead based paint with a new protective coating (Dobrowolski, 1999).

3.7.3 Habitat Maintenance

When a dam safety requirement to lower the reservoir had undesirable environmental impacts, options ranged from removal to full rehabilitation. The preferred solution was selected in part to maintain the reservoir and associated wildlife habitat. Assessment took environmental, social and safety considerations into account along with the economic aspects of the decision. Public and agency input was part of the process. The rehabilitation project has successfully met safety, environment and cultural heritage requirements (Ferguson & Jamieson, 2000).

Following diversion of flow out of a river system, habitat was severely impacted. The construction of a weir acting as an overflow structure provided greatly improved habitats (Fortin, 2003).

Afterbay dams are occasionally located a short distance downstream of a major dam and powerplant, and are designed so that their headpond or reservoir can balance out the operation modes of the powerplant. Peaking power discharges can be leveled out into a more uniform daily discharge downstream of the afterbay dam amongst other benefits, this can protect and enhance habitat along the river system.

3.7.4 Reservoir Ice

The effects of reservoir ice on hydroelectric plants are naturally regional in nature, but are also a function of design and operation. While any dam owner faced with reservoir icing challenges has to develop an appropriate management plan, a number of innovative approaches have been documented.

In an area with severe winter conditions, a hydroelectric plant required at least three tainter gates to be available for winter discharges. Previous solutions to prevent or remove ice included seal heaters, improved seals, bubblers, gate opening cycling and heated water. While innovative, none provided the necessary reliability. An improved design of wall heater eventually proved acceptable (Bockerman & Wagner, 1999).

An improved bubbler system also proved effective in controlling ice build-up against flashboards along a long, low head dam. This system replaced the previously dangerous and labor intensive methods of manually removing ice (Fennelly, 2002).

Intake structures can be vulnerable to ice building up against the trashracks or being entrained in the power flows. Appropriately designed ice sluices and appurtenant works can effectively divert floating frazil ice (Godtland & Tesaker, 1992).

Another solution can be the use of an ice boom, with the design objectives of creating a stable ice cover upstream of the boom, thus reducing ice drifting into the intakes (Abdeinour *et al*, 2000).

3.7.5 Reservoir Debris

Reservoir debris can be of various forms and have safety, environmental and economic impacts on hydroelectric plants. While large floating branches, trees and logs are the most visible forms, aquatic plants, leaves and garbage can create significant impacts.

The sources of reservoir debris can be from forestry practices, slope instability, uncleared reservoir, environmental changes or unrestrained garbage dumps. Dependent on the size and extent of the debris and the design of the hydroelectric plant, spillways can be blocked or damaged, fish screens rendered ineffective and intake trashracks clogged or broken.

Innovative solutions include physical improvements such as larger spillway bays and removal of obstacles, back flushing of fish screens (Locher *et al*, 1993), skimmers for floating garbage removal and a variety of trash rakes. Innovative management solutions include debris management plans that provide an integrated approach to all aspects, including an effective evaluation of the amount of debris to be expected under normal and unusual conditions (Nielsen, 1992).

Current debris management technology, trashrack and spillway designs, raking equipment and their state of art design are covered in an assessment of practice in the USA and Europe (Wallerstein *et al*, 1996).

3.7.6 Water Quality

Reservoir and tailwater quality are directly linked through reservoir operations with a variety of structural and operational improvements available to combat poor water quality. Used together in an integrated management approach, they can provide optimum solutions. Examples of techniques to improve reservoir, release and tailwater quality improvements include aeration, oxygenation, selective withdrawal, turbine venting, destratification and localized mixing (Holland, 1993).

Operation of spillways and sluices at a regulating dam caused increased levels of dissolved gas supersaturation. A new power plant mitigated this impact by reducing spill and incorporating fish friendly turbines (Fortin, 2003).

A fusegate system is not only a cost effective and flexible solution to inadequate spillway capacity, but has significant environmental advantages. Construction impacts are reduced because there is minimal, if any, construction outside the footprint of the dam and spillway, and minimal, if any, disturbance to ground cover or erosion during, and after, construction. Effects on water quality are minimized and sedimentation is controlled. Environmental effects during future operation are also minimized. The system can be designed so that maximum water surface for the new inflow design flood remains the same as for the previous design flood. The result is that no additional land will be submerged (Walz, 2003).

3.7.7 Fish Passage

Over the last two decades there have been significant changes relating to fish passage around hydroelectric plants. Driven by environmental pressures, re-licensing and competing water use, significant research and development activity has resulted in major improvements using innovative approaches (Fortin, 2003), (Turnpenny, 1999), (Helwid *et al*, 1999), (Langeslay *et al*, 2003).

Documented examples include:

- Structurally improved upstream and downstream *bypasses.*
- Physical and CFD *models* to understand hydraulics that fish are exposed to and design improved structural features.
- Plastic molded eel *ladders* designed to allow eels to climb steep grades.
- Fish *screens* located at the upper end of water conduits to divert fish back into the river channel.
- *Acoustic screens* using under water sound stimulus to repel or guide fish into a bypass flow.
- *Fish-friendly turbines* designed to pass fish through the power flow without significant levels of injury or mortality.

3.7.8 Noise Reduction

A noise control package has been developed that specifically targeted areas of turbines, and if required, generator sets with a special acoustic laminate material. This system is of benefit to powerplant staff through better communications and an enhanced working environment (Anon., 1996).

3.8 Management and Knowledge Systems

The approach to management of life extension and modernization for civil works can be greatly enhanced by innovative approaches, particularly knowledge systems. A selection of these includes:

- Management approaches.
- Decision support.
- Guides and standards.

3.8.1 Management Approaches

Life extension, safety and modernization management programs are an integral part of managing the high value assets of a hydroelectric utility. No utility should be without a maintenance management system; usually computerized to identify and manage the routine tasks required to keep the hydroelectric plant operating. However, many utilities take a much more ad-hoc approach to managing capital investments, often relying on a failure or the recommendation of equipment manufacturers or structural specialists.

A modernization management system has been developed comprising four integral parts (Nielsen, 2001):

- Documented process to identify, define and implement improvement.
- Alignment with other systems (technical and financial).
- Decision support software to document business case justifications.
- Training and audit to ensure the system components are in place, are the correct ones and are used properly.

Penstocks, particularly older ones, represent a prime source of risk at hydroelectric plants. A management plan has been developed to manage these risks (Stutsman, 1996).

The features include:

- Preliminary penstock assessment; including documentation, history, inspection, vulnerability assessment, coating, lining and corrosion assessment.
- Secondary penstock assessment; including evaluation of structures, materials, hydraulics, power house and control equipment.
- Rehabilitation, determined by results of assessments.

When utilities are required to find more flexible and environmentally sensitive sources of power, identifying ways to maximize the output from each hydroelectric plant is often considered. A management approach that considers every cost-effective opportunity at each existing project (whether it has a power plant or not) has been adopted (Nielsen & Keir, 1994).

While this program considers all the generation assets that can be modified either structurally or operationally to add value, there is considerable opportunity in the civil structures area. These include:

- Increasing operating head by; raising the dam, incorporating "temporary" heightening or lowering tailwater.
- Reducing hydraulic losses by; lowering friction values in water passages, repair of seals, cleaning trashracks, reducing seepage through dams.
- Diverting water by; diversion canals and channels.

Floating or submerged debris in hydroelectric project reservoirs can affect safety, power production and the use of resources by others. A strategy to consider all aspects of reservoir debris management, has been designed (Nielsen, 1994). The components of the management plan include:

- Sources of debris; type, size and amount that can be expected under usual and unusual (flood) conditions.
- Impact of debris; how debris affects safety, power production and other reservoir uses.
- Mitigation; prevention, removal and disposal.

3.8.2 Decision Support

As facilities age, with limited financial resources and in a competitive market place, decision support tools have been developed to assist dam owners in making optimum decisions on the future of major assets.

A new risk-based decision support has been used to assist in a penstock replacement decision. Initially the condition of the penstock was assessed and the likelihood of failure estimated, using a database that included a large number of failure probability curves. From the estimate of consequences and costs of different failure scenarios, the optimum alternative rehabilitation option was selected (deMeel *et al*, 1998).

An innovative asset management software system has been developed and is in use to provide a full range of decision support for a major hydroelectric utility. This includes budgeting, financial analysis, business case preparation and planning for alternative rehabilitation scenarios (Nielsen & Casey, 2003).

3.8.3 Guides and Standards

There have been many guides issued on the subject of rehabilitation of civil works. Some are formal documented guidelines prepared by groups such as ASCE, ICOLD and its national affiliates, EPRI, etc., while others are less formal guides prepared by individuals or companies. Notwithstanding the proponent, the intent is to formalize a set of procedures, design aspects or processes. Important guides and standards are included as Resources in Appendix A. However, less formal guides which are particularly relevant to this chapter cover:

- *Automated Performance Monitoring of Dams* (USSD, 2002).
- *Improving Reliability of Spillway Gates* (USSD, 2002).
- *Preserving the Historical Significance of Hydro* (Campbell and Dean, 2003).
- *Remotely Operated Vehicle (ROV) Technology: Applications and Advancements at Hydro Facilities* [TR-113584-V7]. 2000 (EPRI, 2002).
- *Assessment and Review of Mechanical and Electrical Equipment for Flow Control* (Barber and Haines, 2002).
- *Instrumentation and Measurements for Monitoring Dam Performance* (ASCE , 2000).
- *RCC Construction for Dam Rehabilitation* (USSD, 2003).

3.9 Technical References

Abdelnour, Razek *et al.* (2000). "Preventing Ice Blockage at Hydro Plant Intakes" in *Hydro Review*, Vol. XIX, No. 1, February, p. 42-53.

Addo, K.O. & S.J. Garner. (2002). "Seismic Rehabilitation of Elsie Dam" in *Proceedings of the Canadian Dam Association Annual Conference, October 6-10, 2002, Victoria, B.C.* Edmonton : CDA. 1p.

Ahlgren, J. *et al.* (1995). "Assessment Methodology for Steel Penstocks" in *Waterpower '95: Proceedings of the International Conference on Hydropower, San Francisco, California, July 25-28, 1995.* Reston, VA. : ASCE. p. 2197-2206.

Alberty, Justin L. (1999). "Asbestos Removal: Dealing with the Challenge in a Hydro Powerhouse" in *Hydro Review*, Vol. 18, No. 1, February, p. 18-20.

Anon. (2002). "Reaching New Heights" in *International Water Power & Dam Construction*, October. p. 42

Anon. (1996). "Reducing Powerhouse Noise" in *International Water Power and Dam Construction*, Vol. 48, No. 7, July. p. 35-36.

Atwood, Robert, *et al.* (2003). "Innovations to Flowline Rehabilitation at PacificCorp Hydro Projects" in *Hydro Review*, Vol. XXII, No. 3, June, p. 46-53.

Aydin, M. & W. Burchat. (2003). "A Simplified Method to Monitor Flows in Large Tunnels" in *International Journal on Hydropower & Dams*, Vol. 10, Issue One, p. 49-52.

Barbour, John & Gord Haines. (2002). "Experience with Assessments of Flow Control Equipment" in *Proceedings of the Canadian Dam Association Annual Conference, October 6-10, 2002, Victoria, B.C.* Edmonton : CDA, 16p.

Bardy, David M. & Bryan F. Gore. (2002). "Deciding to Upgrade Intake Gates Based on Risk Assessment" in *Hydro Review*, Vol. XX, No. 5. September, p. 50-59.

Berggren, A.L. & A. Sandvold. (1995). "The World's First Artificial Ice Plug for a Hydro Tunnel" in *International Journal on Hydropower & Dams*, Vol. 2, Issue Three, May, p. 50-51.

Bockerman, Ronald, W. & Philip A. Wagner. (1999). "Solutions to Spillway Tainter Gate Icing Problems" in *Hydro's Future Technology, Markets and Policy, Proceedings of the Waterpower '99 Conference, July 6-9, 1999, MGM Grand Hotel and MGM Grand Conference Centre, Las Vegas, Nevada.* [CD-Rom]. Edited by Peggy A. Brookshier. Reston, VA.: ASCE.

Bowers, P.W *et al.* (2000). "Searching for Conventional Spillway Control System Alternatives" in *International Journal on Hydropower & Dams*, Vol.7, Issue Three, p. 37-40.

Boyer, Brad B. *et al.* (2001). "Tainter Gate Testing A Holistic Approach" in *Waterpower XII: Advancing Technology for Sustainable Energy, July 9-11, 2001, Salt Lake City, Utah, USA. Technical Papers.* [CD-Rom]. Kansas City, MO.: HCI. 11p.

Campbell, Heather E. & Laura H. Dean. (2003). "Guidelines for Preserving the Historic Significance of Hydro" in *Hydro Review*, May, p. 26-28.

Carrington, Richard, *et al.* (2002). "Des Joachims Generating Station Penstock Transition and Roller Path Rehabilitation Project" in *Proceedings of the Canadian Dam Association Annual Conference, October 6-10, 2002, Victoria, B.C.* Edmonton : CDA,10p.

Caufield, R.J. (2003). "Connection of the Second Manapouri Tailrace Tunnel" in *International Journal on Hydropower & Dams*, Vol. 10, Issue One, p. 53-54.

Charlwood, Robin G. & Zoltan V. Solymar. (1995). "Long-Term Management of AAR-Affected Structures – an International Perspective" in *USCOLD Alkali-Aggregate Reactions in Hydroelectric Plants and Dams Conference 1995*. Denver, CO. USCOLD. p. 19-55.

Charlwood, Robin G. (1995). "Performance Parameters for Management of AAR-Affected Structures" in *Alkali-Aggregate Reactions in Hydroelectric Plants and Dams Conference 1995,* Denver, CO. USCOLD. p. 407-419.

Chouinard, Luc E. *et al.* (2001). "A Proposed Procedure for the Evaluation of the Condition of Spillways" in *CDA 2001 Annual Conference, Fredericton, N.B. Canada, September 30 – October 4, 2001.* Edmonton, Canada. p. 245-255.

Chouinard, Luc E. *et al.* (2003). "A Condition Assessment Methodology for Spillways" in *Waterpower XIII: Advancing Technology for Sustainable Energy, July 29-31, 2003, Buffalo, New York, USA. Technical Papers.* [CD-Rom]. Kansas, MO.: HCI, 12p.

Clarida, Bruce, *et al.* (1999). "The Unconventional Application of Conventional Materials" in *Proceedings of the Canadian Dam Association Annual Conference, Sudbury, Ontario, October 1999*, Montreal, Quebec, Canada. p. 170-174.

Clarida, Bruce. (1999). "Plywood Liner Extends Service Life of Wood-Stave Penstock" in *Hydro Review*, Vol. XVIII, No. 3, June, p. 56-57.

Clark, Bob & Bill Sherwood. (2000). "Searching for Lost Power at Manapouri" in *International Water Power & Dam Construction*, Vol. 52, No. 11, November. p 16-17.

Clarke, Robert O. (2001). "Remotely Operated Vehicle Applications for Long Tunnel Inspections" in *Waterpower XII: Advancing Technology for Sustainable Energy, July 9-11, 2001, Salt Lake City, Utah, USA. Technical Papers.* [CD-Rom]. Kansas City, MO.: HCI, 10p.

Comte, Bernard *et al.* (2000). "Advances in Spillway Design Using Fuse Gates: Application to the Montsalvens Dam" in *ICOLD Twentieth Congress on Large Dams, Beijing.* Paris, ICOLD.

Deans, John J. *et al.* (2003). "Chenaux Generating Station – Tailrace Piers and Wall Rehabilitation Using the Advanced Polymer Encapsulation (APE) System" in *Waterpower XIII: Advancing Technology for Sustainable Energy, July 29-31, 2003, Buffalo, New York, USA. Technical Papers.* [CD-Rom]. Kansas, MO.: HCI, 12p.

deMeel, H.W., *et al.* (1998). "Managing Hydro Assets Rationally: A Risk Based Approach", in *Hydro Review*, Vol. XVII, No. 7.

Denny, M. (1998). "Relining of the Deep Sluices at Roseires Dam" in *International Journal on Hydropower & Dams,* Vol. 5, Issue Three, p. 66-68.

Dohnálek, Jiri, *et al.* (2002). "Defusing the Situation" in *International Water Power & Dam Construction*, October, p. 18-41.

Doujak, E. *et al.* (2001). "A Scoring Mechanism for Gates" in *Uprating and Refurbishing Hydro Power Plants, Prague, December 10-12, 2001.* Dartford, Kent.: International Water Power & Dam Construction. 8p.

Eastman, Keith, *et al.* (2001) "Concrete Expansion – R.H. Saunders GS Innovative Structural Rehabilitation & Mitigation Effectiveness" in *Waterpower XII: Advancing Technology for Sustainable Energy, July 9-11, 2001, Salt Lake City, Utah, USA. Technical Papers.* [CD-Rom]. Kansas City, MO.: HCI. 11p

Edwards, Wayne D & Samuel M. Planck. (1999). "Improving the Reliability of Spillway Radial Gates" in *Hydro Review*, October, p. 32-40.

Edwards, Wayne D. & Samuel M. Planck. (2001). "Spillway Gate Evaluations: A Critical Part of Dam Safety" in *CDA 2001 Annual Conference, Fredericton, N.B. Canada, September 30 – October 4, 2001*, Edmonton: CDA. p. 267-274.

Elfman, Sten & Klas Cederwall. (1999). "Uprating of Old Hydropower Tunnels" in *Uprating and Refurbishing Hydropower Plants Berlin/Germany, May 17-20, 1999.* p. 321-327.

EPRI. (1999). *Hydropower Technology Round-Up Report: Steel Penstock Coating and Lining Rehabilitation.* Palo Alto, CA.: EPRI. [TR-113584-V3].

EPRI. (2002). *Remotely Operated Vehicle (ROV) Technology: Applications and Advancements at Hydro Facilities*. Palo Alto, CA.: EPRI. [TR-113584-V7].

Fennelly, Richard. (2002). "Bubbles Replace Toil and Trouble" in *Hydro Review*, September, p. 60-61.

Ferguson, Keith & Stephen Jamieson. (2000). "Born Again" in *International Water Power & Dam Construction*, April, p. 28-30.

Fishers, J.W. (2003). "Remote Control" in *International Water Power & Dam Construction,* Vol. 55, No. 6, June 2003.p. 50.

Fortin, P. (2003). "Hydro and Fisheries in Canada: Innovative Fish-Friendly Solutions" in *International Journal on Hydropower & Dams*, Vol. 10, Issue 2, p. 88-90.

Gass, Matthew. (2003). "Friction Reduction of Unlined Power Tunnels for Increased Power and Energy Production" in *Waterpower XIII: Advancing Technology for Sustainable Energy, July 29-31, 2003, Buffalo, New York, USA. Technical Papers.* [CD-Rom]. Kansas, MO.: HCI, 4p.

Gilks, Phillip, *et al.* (2001). "A Review and Management of AAR at Mactaquac Generating Station" in *CDA 2001 Annual Conference, Fredericton, N.B. Canada, September 30 – October 4, 2001*, p. 167-177.

Godtland, Klemet & Einar Tesaker. (1992). "Floating Ice As A Problem to Hydropower Intakes" in *Hydropower '92, Proceedings of the 2nd International Conference on Hydropower, Lillehammer, Norway, June 16-18, 1992.* Rotterdam: A.A B, p. 395-402.

Heinrich, Darrell W. (2003). "Relining Conduits with Spincrete" in *Hydro Review*, August 2003, p. 64-66.

Helwig, Lance et al. (1999). "A New Juvenile Fish Bypass System at Bonneville Dam" in *Hydro's Future Technology, Markets and Policy, Proceedings of the Waterpower '99 Conference, July 6-9, 1999, MGM Grand Hotel and MGM Grand Conference Centre, Las Vegas, Nevada.* [CD-Rom]. Edited by Peggy A. Brookshier. Reston, VA.: ASCE.

Holland Jeffery P. (1993). "Overview of Engineering Methods for Reservoir and Tailwater Quality Management" in *Thirteenth Annual USCOLD Lecture: Engineering Solutions to Environmental Challenges*. Denver, Co.: USCOLD. 18p.

Hosko, Mary Ann. "Inspection of a Hydropower Tunnel Using Remotely Operated Vehicles (ROV): A 5-Year Case Study" in *Waterpower '95, Proceedings of the International Conference on Hydropower, San Francisco, California, July 25-28, 1995.* Reston, VA.: ASCE, p. 2645-2653.

Jones, Adam N. *et al.* (2002). "The Role of Underpinning in Hydro Rehabilitation" in *Dams – Innovations for Sustainable Water Resources: 22nd Annual USSD Conference San Diego, California, June 24-29, 2002.* Denver, CO.: USSD, p. 611-620.

Kmetz, M.J. *et al.* (1995). "Modernizing Hydro Gate Hoists for Safety and Economy" in *WaterPower '95, Proceedings of the International Conference on*

Hydropower, San Francisco, California, July 25-28, 1995. Reston, VA.: ASCE, 2590 – 2599.

LaBoon, John H. (2000). "A Legacy Set in Concrete" in *International Water Power & Dam Construction*, Vol. 52, No. 4, April, p. 32,33.

Langeslay, Mike *et al.* (2003). "The Use of Physical and CFD Models to Improve Juvenile Fish Passage at The Dalles Dam, TDA, The Dalles, Oregon" in *Waterpower XIII: Advancing Technology for Sustainable Energy, July 29-31, 2003, Buffalo, New York, USA. Technical Papers.* [CD-Rom]. Kansas, MO.: HCI.

Lardi, Luciano *et al.* (2002). "A New Sealing System for Dams" in *Dams – Innovations for Sustainable Water Resources: 22nd Annual USSD Conference San Diego, California, June 24-29, 2002.* Denver, CO.: USSD, p 529-538.

Larson, Erick. (2003). "Plugging Leaks with Concrete and Plastic" in *Hydro Review*, May, p. 52-78.

Leach, Patrick A. (2003). "Spillway Gate Evaluation and Rehabilitation" in *Waterpower XIII: Advancing Technology for Sustainable Energy, July 29-31, 2003, Buffalo, New York, USA. Technical Papers.* [CD-Rom]. Kansas, MO.: HCI, 12p.

Locher, F.A. *et al.* (1993). "Debris Removal from a Low-velocity, Inclined Fish Screen" in *Thirteenth Annual USCOLD Lecture: Engineering Solutions to Environmental Challenges.* Denver, CO.: USCOLD. 11p.

Locke, Elaine & Ellis D. O'Neil. (1999). "Using Roller-Compacted Concrete to Reconstruct a Dam in Record Time" in *Hydro Review*, Vol. 18. No. 5, August, p. 38-66.

Lupien, Roger. (1995). "Mass Concrete Swelling" in *USCOLD Alkali-Aggregate Reactions in Hydroelectric Plants and Dams Conference 1995.* Denver, Colo.: USCOLD. p. 240-251.

MacTavish, Bruce, *et al.* (2003). "Tainter Gate Trunnion Bearing Testing" in *Waterpower XIII: Advancing Technology for Sustainable Energy, July 29-31, 2003, Buffalo, New York, USA. Technical Papers.* [CD-Rom]. Kansas, MO.: HCI, 9p.

Mason, P.J. (1998). "Recognizing the Effects of AAR at Dams and Hydro Plants" in *International Journal on Hydropower & Dams*, Vol. 5, Issue Five, 1998. p. 90-94.

Mattson, Richard A. & Charles S. Ahlgren. "Nondestructive Examination of Steel Liners and Penstocks to Ascertain Structural Integrity" in *Hydro's Future Technology, Markets and Policy, Proceedings of the Waterpower '99 Conference, July 6-9, 1999, MGM Grand Hotel and MGM Grand Conference Centre, Las Vegas, Nevada.* [CD-Rom]. Edited by Peggy A. Brookshier. Reston, VA.: ASCE.

McDonald, J.E. *et al.* (1997). "Geomembrane Systems for Underwater Repair of Dams" in *Proceedings of Waterpower '97 Conference, Vol. 1*, Reston, VA.: ASCE, p. 175.

Mirza, J. *et al.* (2001). "Special Repairs to the Bersimis-1 Generating Tunnel Walls to Increase Power Production: A Case Study", in *NRC Canada 2001*, p. 411-418.

Mochrie, Bryce N. *et al.* (2001). "Santeetlah Dam on the Cutting Edge of Instrumentation" in *Waterpower XII: Advancing Technology for Sustainable Energy, July 9-11, 2001, Salt Lake City, Utah, USA. Technical Papers.* [CD-Rom]. Kansas City, MO.: HCI. 13p.

Nielsen, N.M. (1992). "Reservoir Debris – Safety, Economic and Environmental Considerations" in *Hydropower '92, Proceedings of the 2nd International Conference on Hydropower, Lillehammer, Norway, June 16-18, 1992,* Rotterdam: A.A.B, p. 425-432.

Nielsen, N.M. (1994). "Less is More: Minimizing Debris in Reservoirs to Maximize Safety" in *Hydro Review*, Vol. XIII , No. 4, May. p. 50-56.

Nielsen, N.M. and G. Keir. (1994). "What's 'Smart' About Improving What You Have?" in *Hydro Review*, Vol. XIII, No. 1. February. p.10-19.

Nielsen, N.M. (2001). "A Modernization Management System for Hydro Plants" in *Uprating and Refurbishing Hydro Power Plants. Prague, December 10-12, 2001.* Dartford, Kent.: International Water Power & Dam Construction. 10p.

Nielsen, N.M. and L. Casey. (2003). "An Asset Planning System for the Energy and Water Business" in *Proceedings of ANCOLD Conference, Hobart, Tasmania, October 28-29, 2003.* ANCOLD, Brisbane. 7p.

Nguyen, Huu Phuong. (2003). "Trends and Technologies for Monitoring Dams" in *Hydro Review*, Vol. XXII, No. 3. June, 2003. p. 54-60.

Regan, Patrick, J. *et al.* (1995). "Penstock Evaluation Using Risk Assessment Methodologies" in *Waterpower '95: Proceedings of the International Conference on Hydropower, San Francisco, California, July 25-28, 1995.* Reston, VA.: ASCE. p. 2187-2217.

Riddle, William H. (1995). "Spillway Dewatering For Replacement of Radial Crest Gates" in *Waterpower '95, Proceedings of the International Conference on Hydropower, San Francisco, California, July 25-28, 1995.* Reston, VA.: ASCE. p. 2694-2703.

Rűesch, C. & L. Lardi. (2003). "Sealing the Deal" in *International Water Power & Dam Construction*, March, 2003, p. 30-31.

Rudolph, Richard M. & James A. Gundry. (1995). "Rehabilitating Old Gates" in *Waterpower '95, Proceedings of the International Conference on Hydropower, San Francisco, California, July 25-28, 1995.* Reston, VA.: ASCE. p. 2583-2589.

Rudolph, Richard M. & Lawrence L. Campbell. (2003) "Spillway Rehabilitation at Dells Hydro: Keys to Economical, Fast-Track Completion" in *Hydro Review*, March 2003, p. 14-19.

Saleh, Kaveh, *et al.* (2002). "Crack Repair in Concrete Dams: Identifying Effective Methods and Materials" in *Hydro Review*, September, p. 40-49.

Schewe, L.D. (1999). "Rehabilitation of an Old German Masonry Dam" in *International Journal on Hydropower & Dams*, Vol. 6, Issue Five, p. 60-63.

Scott, D.L. & D.G. Hill. (2000). "Surveillance Improvements at a Large Canadian Dam" in *Proceedings of the Canadian Dam Association 3rd Annual Conference, September 16-21, 2000, Regina, Saskatchewan.* Edmonton: Canadian Dam Association, p. 203-211.

Scuero, Alberto M. (1995). "PVC Waterproofing Membranes and Alkali-Aggregate Reaction in Dams" in *USCOLD Alkali-Aggregate Reactions in Hydroelectric Plants and Dams Conference 1995.* Denver, CO.: USCOLD. p. 491-505.

Scuero, Alberto M. *et al.* (2000). "New Technologies to Optimize Remedial Works in Dams: Underwater Installation of Waterproofing Revetments" in *Dams 2000.* London, Thomas Telford, p. 298-310.

Sehgal, Chander K. *et al.* (1995). "Proposed Modifications to Hiwassee Dam Radial Gates to Accommodate Concrete Growth on Piers" in *USCOLD Alkali-Aggregate Reactions in Hydroelectric Plants and Dams Conference 1995.* Denver, CO.: USCOLD. p. 529-541.

Shaw, Q.H.W. *et al.* (1999). "Fusegates Enhance Safety and Increase Capacity at Eikenhof" in *International Journal on Hydropower & Dams*, Vol. 6, Issue Six, p. 46-51.

Sonier, A. *et al.* (2001). "Great Falls Sluiceway Rehabilitation" in *CDA 2001 Annual Conference, Fredericton, N.B. Canada, September 30 – October 4, 2001*, Edmonton: Canadian Dam Association, 1p.

Stutsman, Richard D. (1996). "Developing a Cost-Effective Penstock Safety Program" in *Hydro Review*, Vol. XV, No. 4, May, p. 16-23.

Stutsman, Richard, D. (1999). "Penstock Life Extension" in *Hydro's Future Technology, Markets and Policy, Proceedings of the Waterpower '99 Conference, July 6-9, 1999, MGM Grand Hotel and MGM Grand Conference Center, Las Vegas, Nevada.* [CD-Rom]. Edited by Peggy A. Brookshier. Reston, VA.: ASCE.

"Synthetic Aggregate for Abrasion-Resistant Concrete" in *International Journal on Hydropower & Dams,* Vol. 5, Issue Six, p. 86-87.

Szita, Peter, *et al.* (1995). "Slot Cutting Technology in Dams" in *USCOLD Alkali-Aggregate Reactions in Hydroelectric Plants and Dams Conference 1995.* Denver, CO.: USCOLD. p. 383-391.

"Taking a Synthetic Approach" in *International Water Power & Dam Construction,* Vol. 52, No. 1, January 2000, p. 28-29.

Todd, Robert. (2003). "Fatigue Critical Areas in Penstocks at Hoover Dam" in *Waterpower XIII: Advancing Technology for Sustainable Energy, July 29-31, 2003, Buffalo, New York, USA. Technical Papers.* [CD-Rom]. Kansas, MO.: HCI, 6p.

Townshend, P.D. (2000). "Towards Total Acceptance of Fully Automated Gates" in *Dams 2000.* London: Thomas Telford, p. 81-94.

Turnpenny, A.W.H. ((1999). "Mitigation Measures for Migratory Fish at Hydro Plants" in *International Journal on Hydropower & Dams,* Vol. 6, Issue Five, 1999, p. 83-88.

United States Society on Dams (USSD). Committee on Monitoring of Dams and Their Foundations. (2002). *RCC Construction for Dam Rehabilitation*, Denver, CO.: USSD, March 2003.

United States Society on Dams (USSD). Committee on Monitoring of Dams and Their Foundations. (2002*). General Guidelines for Automated Performance Monitoring of Dams*. Denver, CO.: USSD, November 2002.

Veilleux, Mario. (1995). "Hydro-Quebec's Experience Using Deep Slot Cutting to Rehabilitate Concrete Gravity Dams Affected by Alkali-Aggregate Reaction" in *USCOLD Alkali-Aggregate Reactions in Hydroelectric Plants and Dams Conference 1995.* Denver, CO.: USCOLD. p. 221-235.

Wagner C.D. & J.K. Meisenheimer. (1997). "Developing a Management Plan for Alkali-Aggregate Reaction in Dams" in *Proceedings of Waterpower '97 Conference*, Vol. 1, p. 274.

Wallerstein, N. *et al.* (1996). "Debris Control at Hydraulic Structures in Selected Areas of the USA and Europe" in *Wallerstein, N. Thorne, C. & Abt, S. (1996). Debris Control at Hydraulic Structures in Selected Areas of the USA and Europe.* Nottingham University, Dept. of Geography: Nottingham. 114p.

Walz, Arthur H. Jr. (2003). "Analyzing Solutions: Methods of Increasing Spillway Capacity" in *Hydro Review*, May, p. 46-51.

Westermann, Jerry. (1998). "Expandable Gates Can Be Used in Various Water Passages" in *Hydro Review*, August. P. 122-123.

Westermann, Gerold D. (1999). "Penstock Rupture: Assessing the Risks, Identifying Potential Problems" in *Hydro Review*, July, p. 76-83.

Yadon, D.M. *et al.* (1998). "RCC for a Major Spillway Rehabilitation in Severe Climatic Conditions" in *International Journal on Hydropower & Dams*, Vol. 5, Issue Three, p. 35-40.

Yamamoto, Kosuke *et al.* (2003). "Latest Maintenance Technologies for Spillway Gates and Steel Penstocks in Japan" in *Waterpower XIII: Advancing Technology for Sustainable Energy, July 29-31, 2003, Buffalo, New York, USA. Technical Papers.* [CD-Rom]. Kansas, MO.: HCI, 13p.

4.0 CHAPTER 4 – STRUCTURES

4.1 Introduction

Chapter 4 introduces the perspective of how real problems associated with structures are identified, defined and implemented to extend the life, or upgrade, a hydroelectric project or dam. The discussion is oganized by principal project feature, namely intakes, dams, spillways and reservoirs, powerhouses, fish passage, trashracks and trashrakes. Tables 4.2-1 through 4.7-1, located at the end of this subsection, summarizes by feature, the issues covered, opportunities for life extension and upgrade, and case histories.

The common approach to this guideline has been to document numerous means and methods to upgrade, or extend the life of, civil works associated with a project while acknowledging that the selection of the most promising solution is very project dependent. These guidelines are written as a reference tool to assist in this selection, and not as a prescriptive design aid. Moreover, these guidelines are based on experience from existing works, and do not purport to cover all issues that could be associated with civil works.

The definition of life extension and upgrade as used in these guidelines is as follows:

Service Life Extension - Activity that extends the life of the civil feature beyond that which would be expected with normal maintenance (make it last longer).

Upgrade - Activity that improves performance of the civil feature beyond the current performance (make it work better). Exchange of a system or component with a similar or different system may not necessarily be considered an upgrade.

As background, Chapters 1 and 2 describe the processes to extend the life and upgrade hydroelectric civil works, and outline the steps to better understand the issues, identify opportunities, and recognize limitations. These chapters also provide insight on understanding the existing conditions and evaluating proposed changes that include identification and selection of the preferred alternative.

Chapter 3 is a review of innovative technologies that cover civil aspects of hydroelectric projects. These technologies have been developed mainly to reduce costs and improve profitability, reliability and environmental performance. Chapter 3 has been written around seven activities associated with civil works, whereas Chapters 4, 5 and 6 are focused specifically on the civil structures, water conveyances and water control devices, respectively.

This chapter describes examples and solutions as a way to illustrate techniques for improving the performance, or extending the service life, of civil structures. Also provided in this chapter is general broad-based information, or "rules of thumb", to allow the reader to assess if, or when, a structure has reached the end of its service life, or if the structure warrants, or is capable of, improvements to its performance.

The common format used in Chapters 4, 5 and 6 starts with descriptions of the function of specific types of civil features, their problems and limitations, possible corrective measures and alternative solutions for life extension or upgrade. Not all aspects of the features will be discussed fully. Information on each feature is provided by subsection as follows:

a) Function.
b) Problems.
c) Corrective Measures.
d) Opportunities.
e) Case Histories.
f) References.

Function:

A brief description of the function or purpose of the feature is provided. Some civil features may have multiple uses as part of the hydroelectric project. Depending on the actual use and functions of the structure, its expected service life and opportunities for improvement can be determined.

Problems:

Typical problems and limitations (multiple if applicable), and the (root) causes are identified. If the civil feature has an operating system, a description of the limitations of the system is provided. Some problems are presented as a simple list and others are described in detail. General, broad-based "rules of thumb" regarding the service life, design or operational performance of a civil feature are provided as applicable. These rules of thumb are intended to provide guidance as to whether a feature has reached the end of its life, or the feature and its function could be improved.

Corrective Measures:

Options are identified that have been used (or considered for use) to rectify the problems identified. For solutions with multiple alternatives, the pros and cons of the alternatives are discussed. However, the solution identified may be site specific, and may not be applicable to all similar problems.

Opportunities:

Possible opportunities (additional benefits) are identified for upgrading (improving) the civil feature beyond that associated with just addressing a problem or limitation of service life.

Case Histories:

Detailed examples are provided to describe typical solutions that have been used to address "real life" problems associated with that type of feature. In some of the examples the solutions may have extended the service life of a civil feature, improved its performance, or resulted in both an extension of service life and improvement of performance.

Each case history is structured to provide the following information:

- Background to the project and the function of the civil feature (background).
- Problems and causes (problem).
- Corrective measures and selected alternative (solution).
- Opportunities and benefits (results) provided to the owner as a result of life extension or upgrade.

References:

There are three methods used in each Chapter to support the body of knowledge presented in the guidelines.

- Collective Knowledge.
- Technical References.
- General Resources.

Collective Knowledge is a collection of references considered by the guideline authors to be primary references that describe the function, operation, or design of each civil feature. Some of the references also provide information pertaining to inspection and assessment of the feature, problems, causes and possible solutions. The references are located at the end of each section to which they are pertinent.

Technical References are references specifically identified in the text of these guides, designated by name and date (i.e. ASCE, 1995) with the full reference at the end of each section.

General Resources include any background resource that is deemed to have value for broader reference and additional reading. It includes learned societies, government agencies and other sources. The General Resources Library is found in Appendix A.

Table 4.2-1 Intakes

Issues or Problems Covered

- Inadequate original design
- Change in performance criteria
- Deterioration of project components
- Hydraulic performance – head loss, approach velocities, model configurations, velocities, vortices, uneven distributions and levels
- O&M difficulties: rock trap cleanout, debris, sheet and frazil ice, sediment
- Downstream environmental issues – dissolved oxygen, water temperature

Opportunities for Solutions, Life Extension and Upgrade

- Vortex suppression to improve performance
- Intake channel modifications to improve flow characteristics
- Multi-level intake towers for water temperature control
- Guide walls and guide vanes
- Sluicing of debris, ice and sediment
- Head loss reductions, hydraulic improvements and capacities for increased generation

Table 4.3-1 Dams, Spillways and Reservoirs

Issues or Problems Covered	
• Inadequate original design • Change in performance criteria – adequacy of capacity and seismic design • Deterioration of project and material components • Structural integrity and stability issues • Performance of flow surfaces • Non-functioning or non-existent drains • Alkali-aggregate reactivity (for concrete structures) • Leakage – through joints and membranes	• Loss of material – timber crib dams • Foundation uplift • Hydraulic fill dams – liquefaction • Lack of flood proofing • Erosion of channels • Seepage through or beneath earth fill dam • Foundation drain cleaning • Buttressing to improve seismic resistance • Foundation grouting to reduce seepage
Opportunities for Solutions, Life Extension and Upgrade	
• Alkali-aggregate reactivity remediation, slot cutting • Increase spillway discharge via structural improvements: new or additional spillway, raise dam or add crest wall • Provide overtopping protection with grass, geotextiles, mattresses, gabions, riprap, concrete and concrete blocks, RCC and soil cement • Lower or lengthening spillway crest • Providing aeration to reduce cavitation on surfaces • Changing spillway operating characteristics to extend life • Air bubbler system to reduce ice loading • Applying corrosion repair materials • Epoxy injection for cracking	• Controlling spillway crest level with inflatable system • Ultrasonic testing to locate voids in spillway slab • RCC surfacing of spillway • Anchoring to improve stability to meet design criteria • RCC mass for stability • Drains and grouting to reduce uplift • Armoring of spillway channel • Early notification systems for major flood events • Relining and repair with geomembranes • Sheet pile cutoffs • Toe berms

Table 4.3-1 Dams, Spillways and Reservoirs (continued)

Table 4.3-1 Dams, Spillways and Reservoirs (continued)

Case Histories (continued)

Table 4.4-1 Powerhouses

Issues or Problems Covered	
• Inadequate original design • Change in performance criteria • Foundation stability: settlement, heave • Structural stability: seismic, hydraulic • Structural damage: cracks, aging, poor maintenance, freeze-thaw, alkali-aggregate reactivity	• Seepage into powerhouse • Roof damage: aging, poor maintenance • Damage to water passageway: corrosion, cavitation, vibration • Deterioration of project components
Opportunities for Solutions, Life Extension and Upgrade	
• Post tensioned anchors • Pressure relief wells to drain foundations • Waterproof coating of water passage wall to reduce seepage • Powerhouse bridge crane and rail support upgrades • Reduce maintenance costs	• Operating procedure improvements for civil works monitoring • Civil works modifications to accommodate new equipment or equipment damaged by alkali-aggregate reactivity

Table 4.5-1 Fish Passage Facilities

Issues or Problems Covered

- O&M of fish passages
- Upstream and downstream passage
- Downstream passage increased predation
- Injury/mortality, increased predation
- Habitat alteration

Opportunities for Solutions, Life Extension and Upgrade

- Transportation
- Screens
- Structural guidance
- Behavioral devices
- Turbine passage
- Fish pump exists
- Spillway releases
- Fish ladder
- Fish lifts
- Trap and haul
- Fish pumps
- Attraction flow
- Fishways

CASE HISTORIES **PAGE NO.**

Table 4.6-1 Trash Racks

Issues or Problems Covered

- Corrosion
- Inadequate design
- Structural damage and deterioration
- Frazil and anchor ice
- Hydraulic loading
- Vibration
- Zebra Mussels

Opportunities for Solutions, Life Extension and Upgrade

- Reduce head loss for increased generation
- Reducing maintenance costs for trash raking
- Excluding fish and eels
- Reduce approach velocities
- Corrosion resistant racks and coatings

Table 4.7-1 Trash Rakes

Issues or Problems Covered

- Maintenance issues: damaged rake, worn out, alignment, damaged mechanical parts
- Reduced efficiency or generation and rake performance
- Personnel Safety
- Inadequate lifting or sluicing capacity
- Need to dispose of large quantities of trash
- Increasing cost for debris removal

Opportunities for Solutions, Life Extension and Upgrade

- Reduce O&M cost – automated operation
- Reduce head loss for improvement in generation
- Improved safety of personnel
- Prevent tripping due to differential head losses

4.2 Intakes

4.2.1 Function

The function of an intake at a hydroelectric project is to divert water from a source such as a river, reservoir or forebay under controlled conditions into the penstock(s) or conduit(s) leading to a powerplant. Most intake structures house trashracks that prevent large debris and ice from entering the water passages together with gates, or valves, for controlling the flow of water, and for dewatering of the intake for maintenance purposes.

The subject of intakes has been discussed in detail in the *Guidelines for Design of Intakes for Hydroelectric Plants*, (ASCE, 1995) and in the *Guidelines for Rehabilitation of Civil Works of Hydroelectric Plants*, (ASCE, 1992). Much of the information given below is derived from these publications. Trashracks and trashrakes are discussed in more detail in Sections 4.6 and 4.7.

There are two different types of intake structures depending on the function of the conduits being supplied. Power intakes are defined as those intakes directly supplying turbines by means of pressure conduits. Conveyance intakes are defined as those intakes which usually do not directly supply turbines but supply other conveyance structures such as canals, flumes, or tunnels, which in turn supply power intakes. Conveyance intakes are usually followed by power intakes at the point of connection with the pressure conduits.

Intakes are designed to deliver the required flow over the desired range of headwater elevations with maximum hydraulic efficiency. Design requirements for intakes are based on siting, operational, geologic, structural, hydraulic, environmental, and economic considerations. The wide variety of constructed intake arrangements reflects the differing considerations encountered from site to site.

An intake structure must be structurally stable, even when dewatered; the velocity through the trashracks, gates and other passages must be confined within economic and practical limits; the water passages must be shaped so that transformation of static head to conduit velocity is gradual, with minimum eddy losses, and minimum head losses; the design should inhibit the formation of vortices at the intake; and all equipment should be reliable, simple to operate and easy to maintain.

The intake structure must be adequate to resist all required loading conditions. This includes overall stability both in the watered and in the unwatered condition. Loads to be considered include dead and live loads, equipment loads, hydrodynamic, hydrostatic, soil, seismic, ice and snow loading as applicable.

An intake should be designed to minimize the hydraulic entrance loss (head losses) caused by acceleration of the water and eddy losses at the trashracks and gate guide recesses. This is achieved by limiting the trashrack velocity and minimizing acceleration of the water to achieve a smooth rate of acceleration. The intake should not entrain air that can lead to problems with turbine operation – producing vibration and loss of power. Vortices should be avoided at the entrance as they entrain air and may cause hydraulic instability. Trashracks should not be exposed, even during low forebay levels and the intake gate lintel should be submerged below minimum forebay level to minimize potential problems caused by air entrainment, vortices or freezing. Approach flows to the intake should preferably be perpendicular to the trashrack face.

Problems caused by river bed load and silt can have a major impact on the design of the intake in streams where mobile bed conditions exist.

Air vents are typically incorporated in the intake structure and are intended to prevent collapse of the penstock due to excessive vacuum when closing the intake gates.

Environmental considerations affecting intake design include fish entrainment and mortality, upstream and downstream fish passage, minimum instream flow releases, trash handling, and water quality considerations including temperature and dissolved oxygen.

Instrumentation and control requirements depend on the individual plant and its method of operation. Instrumentation at the intake may include a water temperature sensor to detect the onset of frazil ice; a trashrack pressure differential sensor to detect any restriction or blockage of the trashrack; an intake water level sensor to measure headwater level; a gate water velocity sensor to detect abnormally high velocities indicating rupture of the downstream conduit; a conduit water pressure sensor to detect when the gate has closed so that the conduit can be dewatered; a gate-opening position indicator to signal the position of the gate; a hoist house interior temperature sensor to initiate an alarm indicating heater failure; and/or an intruder alarm at the hoist house to signal unauthorized opening of a door or window.

4.2.2 Problems

The power intake is a critical component of a hydroelectric generating facility. Problems may result from the structural condition of the intake, from hydraulic conditions, or from environmental considerations. Key issues are inadequate original design, change in performance criteria, deterioration of intake structure components and need for environmental mitigation measures.

The overall structural adequacy of the intake structure may not be acceptable due to deterioration of materials over time or the changing of loading conditions from those originally envisioned.

Excessive head loss in the intake reduces generation and, for very low head projects, can move the operating point for the turbine to a position of lower efficiency. Air entraining vortices may result in rough operation of turbine machinery, in addition to increasing head losses. At low head installations with horizontal axial flow turbines and an improperly designed intake can create an uneven flow distribution at the turbine runner, adversely affecting performance.

Hydraulic considerations include changes in hydrology and operating policies (e.g. minimum flow releases, restrictions in impoundment fluctuations), as well as the ability of the intake to accept flows that are different from the original design flows. Flow through the intake may be greater than originally designed if installed capacity has been increased, or may be less than originally designed if bypass flows or other environmental flows have been instituted since the original constitution.

When considering hydraulic design changes, issues to be evaluated are changes in hydrology or operation policy, submergence, head losses, approach flow conditions (e.g. flow patterns, velocity profiles, fish protection, fish impingement and vortices), trash, sediment (increase or decrease), ice (increase or decrease), and location of the intake with respect to wind and approach of flows.

Modifications to the intake may be necessary to provide for the up-to-date environmental provisions that are necessary for both fish and wildlife, recreational opportunities, cultural, aesthetic, and other aspects of environmental quality.

4.2.3 Corrective Measures

Structural repairs may be necessary for the intake structure, including any approach or training walls. This may extend from concrete repair and replacement to complete replacement of the intake structure. Methods used are described elsewhere in this guideline. It should be noted that if the use of a cofferdam is required to dewater the intake structure, then this could be a significant permitting, scheduling, and cost item.

Many older intakes, or intakes where flow is being increased as part of a plant upgrade, have problems due to poor approach channel conditions with resultant adverse effect on turbine performance or due to vortex formation at the intake entrance. The provision of splitter walls or vanes in the intake approach channel may be a solution to address approach problems. For vortices, the provision of hoods or vortex rafts at the intake can be considered.

Intakes are intended to provide a smooth efficient transition from the forebay to the water conduits. Problems could occur with head losses and turbine performance, if the intake is not properly oriented, designed, or operated. The life cycle costs of upgrading an existing intake to remedy this should be compared to those for constructing a new intake, particularly if the structural integrity of the existing intake is poor.

Correction measures to address sediment issues may include provision of sediment excluders or means to sluice (release) sediments. To address accumulation of floating ice or debris, a sluiceway may be required.

Environmental corrective measures may include exclusion devices in the case of fish entrainment and mortality, and fish lifts or ladders.

4.2.4 Case Histories

**No. 1 High Cost of Intake Operation and Maintenance
Naches Project (ASCE, 1995)**

The Naches Hydroelectric Project is located in Yakima County, Washington. The conveyance intake at the project delivers a maximum of about 550 cfs to a canal serving two powerhouses and numerous irrigators. An evaluation of the intake revealed several deficiencies, which produced operation and maintenance headaches. The intake before and after modification is shown in plan in Figure 4.2-1.

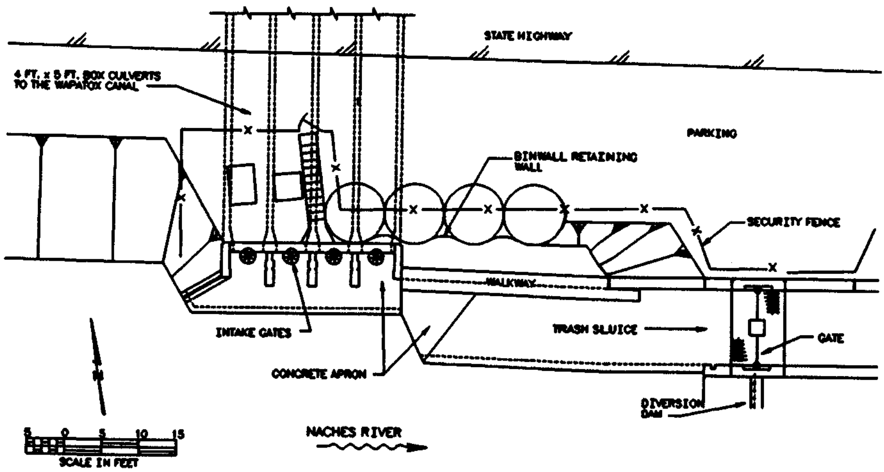


Figure 4.2-1a Naches Hydroelectric Project – Before Modification (courtesy of ASCE)

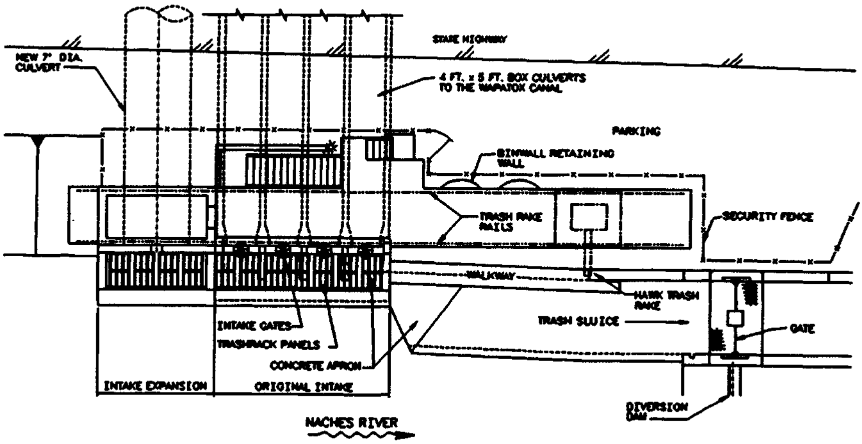


Figure 4.2-1b Naches Hydroelectric Project – After Modification (courtesy of ASCE)

The existing intake gates did not seal properly, and the gate operators were too low to be used during high flows. The existing intake had no trash racks. Coarse sand passed easily through the intake, requiring frequent cleaning of the rock trap at the canal entrance. Debris lodged in and against the gate slots, creating unacceptable head losses and making gate operation difficult. Flow velocities through the existing conduits were high enough to make the handling of coarse sediment and debris without trash racks difficult. Both frazil and sheet ice frequently formed in the river during the winter months, and handling ice by hand was both difficult and dangerous.

Various modifications were planned and constructed to overcome these operation and maintenance difficulties. New gates were installed in the existing gate openings. The gate operators were located on a new operating platform constructed above high water level. A soft ground tunnel parallel to the existing conduits was bored through the adjoining highway and lined to provide additional flow conveyance and thus reduce flow velocities. A gate was installed on the upstream end of this new conduit. Trash racks were installed in front of both the existing and new gated sections. A deck was cast in front of all the conduits, and equipped with rails to accept a backhoe-type trash rake. A trash rake, capable of handling both debris and ice, was placed in service. Lights were installed to facilitate night operations.

The changes made to the intake were effective in reducing the operation and maintenance problems experienced with the control of sediment, debris and ice.

No. 2 Increased Hydraulic Capacity
Mill "C" Project (ASCE, 1995)

The Mill "C" Hydroelectric Project is owned and operated by the New York State Electric and Gas Company (NYSEG) and is located on the Saranac River in northern New York. The original plant had an installed capacity of 2.25 MW (two turbine/generators), and 65 feet head with a plant flow of 500 cfs. The powerhouse was supplied with water from the Mill "C" dam through a steel penstock 400 feet in length. Following analysis of the flow records of the site, it was decided to upgrade the plant by the addition of a third unit of 3.5 MW at 64-foot head, and to replace the steel penstock with an 11.5-foot diameter fiberglass penstock. The new plant flow is 1,300 cfs.

The problem was how to pass the new flow through the existing intake without creating poor hydraulic conditions, and thus adversely affecting performance.

Various schemes were studied for improving inlet conditions at the Mill "C" dam intake to accommodate the increase in plant flow from 500 cfs to 1300 cfs. As part of these studies, a physical hydraulic model was commissioned. An undistorted Froude model with a length scale of 1:18 was chosen. The use of a vortex suppression raft at the existing structure, the addition of guide vanes at the penstocks, coupled with the extension of the penstocks, and a new intake structure specifically designed for the higher flows were modeled. The vortex suppression raft (existing intake) and a new intake structure were found to be acceptable.

Both a vortex suppression raft (existing intake) arrangement and a new intake structure were bid with the overall power project civil construction contract. The new intake was selected on the basis of cost. Since commissioning, the new intake has performed well and vortex formation has not been observed.

No. 3 Hydraulic Approach Modifications
Upper Mechanicville Project (ASCE, 1995)

The Upper Mechanicville Hydroelectric Redevelopment Project is owned and operated by the New York State Electric and Gas Corporation (NYSEG). It is located on the right bank of the Hudson River at Barge Canal Lock and Dam C-3 in the Town of Stillwater, New York. The lock is on the opposite abutment of the dam.

The site was formerly part of a paper mill, which used the head created by the dam to generate hydropower for its operations. Water was supplied to the original units through a power canal along the right bank with flow into the canal controlled by a head gate structure at the right abutment of the spillway. The total discharge capacity of the units was 5,400 cfs, which is only a small part of the available flow. The new development proposed at Mechanicville consisted of the removal of the existing station and the reconstruction of the powerhouse incorporating two 8.5 MW vertical double-regulated Kaplan turbines. The total installed capacity of the redeveloped project is 17 MW at a total rated discharge of 12,000 cfs and a head of 19 feet. This significantly altered the flow regime upstream and downstream of the dam.

With the new flow regimes upstream and downstream of the dam, and with the existing upstream approach and downstream tailrace channel configuration, flow conditions would have seriously affected both navigation and turbine performance.

Two comprehensive physical hydraulic models (one for the upstream reservoir and intake channel to the powerhouse, and one for the downstream tailrace and navigation channel) were constructed to evaluate the hydraulic effects and improve prototype conditions where necessary. Both were undistorted Froude models with a length scale of 1:60.

Approximately 70 percent of the river flow is concentrated on opposite river banks to the proposed plant intake. The resulting curvilinear approach flow had the potential to cause separation, eddies, and vortices in the intake channel. This complex three-dimensional flow, the potential problems, and their mitigation were investigated in the physical model. Several intake channel configurations were tested. By making the following modifications to the channel, a satisfactory velocity distribution was obtained for the vertical turbine arrangement:

- A guide wall approximately 80 feet in length was formed on the left side of the intake channel by retaining three of the powerhouse construction phase cofferdam cells. This guide wall provided sufficient distance from the point of flow separation to the powerhouse so that the main flow could reattach to the channel wall.
- Two partial-depth guide vanes were constructed in the entrance channel (east and west guide walls) to help reduce the variance of the velocity distribution.

Testing of the downstream flow patterns indicated that the tailrace channel should be approximately 900 feet long. In order to prevent high velocity crosscurrents at the navigation channel, a bi-level channel was constructed. The tailrace channel invert elevation on the left side is about 3.5 feet higher than on the right side. This creates lateral flow out of the channel upstream, which achieves a downstream component prior to reaching the navigation channel, and thereby reduces the objectionable cross currents. The deeper right side of the channel, which discharges further away from the lock approach, permits higher velocities, which allows the powerhouse to discharge water without undue tail water effects.

The project has performed well since commissioning with good approach conditions and low tail water channel losses.

No. 4 Control of Water Temperature
Shasta Dam (USBR, 1997)

Shasta Dam and Lake are located on the Sacramento River about 15 miles north of Redding, California. The Sacramento River and Shasta Dam/Lake are part of the Central Valley Project (CVP), which was developed and is managed by the U.S. Bureau of Reclamation (USBR). Shasta stores some 2.8 million acre-feet, or about 40 percent of CVP water supplies, and generates about 2 billion KWh, or about 42 percent of CVP hydropower.

In order to provide cool water for the salmon fishery, USBR has, since 1987, been voluntarily bypassing Shasta's power plant during hot weather, at a cost approaching $40 million over the years in replacement power. In 1989, USBR engineers began researching a means to control the water temperature at Shasta Dam while maintaining revenue from power generation.

A multi-level intake structure, known as the Temperature Control Device (TCD) was designed and installed over the existing power penstock intake structures and a low-level intake structure located on the upstream face of Shasta Dam. There are five 15-feet diameter power penstocks (intake centerlines are at elevation 815) that serve the existing powerplant. Construction of the TCD to provide selective-level withdrawal capability to the existing penstocks was completed in 1997. The TCD is a steel structure with two main parts. Part of the structure covers the power intakes and part covers the low-level intake. High-level withdrawal, at or above the existing power intakes elevation, is controlled by a 250-foot wide by 300-foot high section that encloses all five existing power penstock intakes. This section is divided into three vertical compartments, one per power intake, and each compartment has three openings with hoist-operated gates and trash racks to allow selection of the reservoir withdrawal level. The 125-foot wide by 170-foot high low-level structure, located to the left of the main power intakes, acts as a conduit extension to access the deeper, colder water near the center of the dam. The TCD is designed for a discharge capacity of 19,500 cfs and has a reservoir operating range between elevations 840 and 1065. The TCD consists of 9,000 tons of structural steel and metal work.

This massive gated steel device provides selective withdrawal capability for the power penstocks to draw water from the reservoir's surface during winter when the water is suitably cool, or from deep on the reservoir during summer when the surface water is too warm.

No. 5 Seismic Upgrade of Low Level Outlet (LLO) Elsie (BC Hydro, 2004)

BC Hydro's Elsie Dam, 25 miles (40 km) north-west of Port Alberni in central Vancouver Island, British Columbia, was constructed in 1958 across Ash River for the 27 MW Ash River Generating Station. The Maximum Normal Reservoir Level of the Elsie lake is El. 1085 ft (330.71 m). Water from the reservoir is diverted through an intake to the generating station that is located on the north shore of Great Central Lake. The Elsie dam consists of a Main Dam, four Saddle Dams, a free overflow Spillway and a Low Level Outlet (LLO) under Saddle Dam 1. During construction, the LLO was used for diversion and after construction it was being used primarily for fishwater releases.

Deficiency Investigations for the Elsie Dam confirmed that earthquake induced loading at relatively small earthquakes:

- Would cause the LLO intake tower to topple.
- Could fracture the corroded LLO steel conduit and its under-reinforced concrete encasement, causing piping through the downstream shell leading to possible failure of Saddle Dam 1.

Either of these events limit the reservoir drawdown capability of the facility, which is required to reduce risk of post-earthquake dam failure due to increased seepage.

Prior to the seismic upgrade works, the LLO consisted of a concrete intake tower with a trashrack, a concrete-encased steel conduit and a Hollow Cone Valve (HCV) at the downstream end as a primary control for regulating discharge.

The following improvements are being implemented to upgrade the Low Level Outlet:

- Installation of an intake vertical lift gate, for upstream flow control, with a larger trashrack to increase discharge capacity of the LLO.
- Replacement of the existing concrete intake tower of the LLO with a tubular steel tower.
- Installation of a new liner inside the existing conduit and extension of the LLO conduit by approximately 64 ft (19.5 m) to accommodate downstream berm extension of Saddle Dam 1.
- Refurbishment of the Hollow Cone Valve by replacing and repairing its mechanical equipment, cleaning and recoating the valve, and installation of a HCV actuator and control panel.
- Installation of a Butterfly Valve (BFV) just upstream of the HCV for secondary flow control.
- Installation of a Diesel Generating set.
- Construction of a larger LLO building and outlet structure to accommodate the Butterfly Valve, Diesel Generating set and other additional electrical, P & C and mechanical equipment.

All the upgrades at Elsie LLO will be implemented by October 2004. With these upgrades, the LLO will meet current seismic design criteria of the Canadian Dam Association - Dam Safety Guidelines, and provide additional reservoir drawdown capability required to reduce the risk of post-earthquake dam failure due to increased seepage.

Additional benefits from the LLO upgrade are:

- The BFV can be operated under full flow conditions to isolate the HCV for repair or maintenance, or to control flow in an emergency.
- The upstream vertical lift gate at the LLO intake is available to isolate the LLO conduit from the reservoir. However, this gate can only be operated under no-flow conditions. By closing the HCV, lowering the intake gate and dewatering the conduit, the BFV could also be isolated for repair or maintenance.
- With the LLO intake gate closed and removing the BFV, the LLO conduit can also be inspected for any post earthquake damage and repair.

The existing LLO, the proposed upgrades and photographs taken during the construction of the improvements to the LLO are shown in Figure 4.2-4 and 4.2-5.

Figure 4.2-2 New Intake Tower, Gate and Trashrack Installed Upstream of LLO, Elsie Dam (courtesy of BC Hydro)

Figure 4.2-3 New Conduit Liners Before Installation Inside the Old Liner (courtesy of BC Hydro)

4.2.5 Collective Knowledge

1. American Society of Civil Engineers (ASCE). (1992). *Guidelines for Retirement of Dams and Hydroelectric Facilities*, ASCE, New York, NY.
2. American Society of Civil Engineers (ASCE). (1995). *Guidelines for Design of Intakes for Hydroelectric Plant*, ASCE, New York, NY.
3. American Society of Civil Engineers (ASCE). (1992). *Guidelines for Rehabilitation of Civil Works of Hydroelectric Plants.* ASCE, New York, NY.
4. Murray, D.G. & Gordon, J.L. (1985). "Intake Design: Concepts to Minimize Cost and Maximize Output" in *Hydro Review,* Spring 1985.
5. United States Bureau of Reclamation (USBR). (1970). "Design of Small Dams" in *Water Power, April 1970.*
6. Gordon, J.L. (1970). "Vortices at Intakes" in *Water Power, April 1970.*

4.2.6 Technical References

ASCE. (1995). *Guidelines for Design of Intakes for Hydroelectric Plants.* American Society of Civil Engineers, New York, NY.

BC Hydro. (2004). Project files for BC Hydro, Vancouver, British Columbia.

4.3 Dams, Spillways, and Reservoirs

4.3.1 General

Though relatively simple in purpose, water retaining structures are as diverse as they are complex. They take on many names, shapes and sizes, and are constructed of numerous materials including soil, rock, concrete, masonry, wood, and steel. With all these variables, their main purpose remains the same to retain water.

It may seem odd that spillways are included with the water retaining structures since their very name implies the release of water. Spillways, however, do more than release water. They actually *regulate* the release of water, but in order to do that, they must first impound the water and hence, are considered water retaining structures. As you will see, many of the problems associated with non-overflow structures; i.e., traditional dams, are also common to spillway structures.

Gates and valves are not discussed in this section, although they too serve as water retaining structures. They are included in Chapter 6, as their purpose in a project is to control the water rather than to simply retain it, and they are a feature that is typically attached to a larger water retaining structure.

Over the last two centuries, tens of thousands of dams and spillways have been constructed in the United States for a variety of purposes and needs, including water supply, flood control, recreation, and hydraulic power. Many early structures were constructed for small mills and factories. For the past two or three decades, relatively few dams have been built domestically for the purposes of power generation, though significant activity related to spillway enhancements and upgrades has taken place.

Dam engineering has evolved as much on the basis of trial and error process as it has on the use of science. The design of older dams was part science, part experience and sometimes guesswork. Often, if a particular design was successful at one site, it was used as the model for construction at other sites. If a design or method of construction failed, it was modified and tried again. Experience does not necessarily tell you what will work, but it will tell you what will not work.

With advances in science, engineering and construction techniques, and changes in regulatory requirements and society's goals, the functions of dams, both new and old, are changing. Some old dams have been demolished as they were no longer serving the needs of society or were deemed unsafe. More often than not however, old dams are rehabilitated in some way, or another, to extend their useful life. For example, older mill dams and their reservoirs are being converted to recreation facilities and water supplies. The mills themselves are often considered historical structures and are preserved for purposes not originally intended, such as museums, condominiums or office buildings.

4.3.2 Concrete Dams & Spillways

a) Function

The primary function of a water retaining structure is to retain, divert or control water for the purposes of irrigation, water supply, flood control, and hydroelectric power generation. Fundamental to the successful performance of these structures is an appropriate foundation design. This is not to say that perfect foundations are required in order to construct them, rather the design of the structure must accurately account for and address the specific foundation characteristics at a given site. In actuality, the foundations at many locations are inherently worse than surrounding areas, which is why the river channel exists in the first place. That is, except for the cases of narrow valleys with significant contour relief, the river channel has formed due to the fact that the materials through which the water flows are the most easily eroded. Dams, like bridges, are typically constructed in locations with adverse geological features which require special consideration during design and construction.

There are three fundamental issues related to foundation design for dams: bearing capacity, sliding resistance, and seepage. Insufficient bearing capacity can result in structural failure of the foundation material and unacceptable deformations of the dam. Adequate sliding resistance is required to maintain the integrity and safety of the dam (i.e., keep it where it was intended). Seepage through the foundation, or through the dam body itself, can result in weakening of the foundations, and ultimately, failure of the structure, as well as loss of reservoir.

Foundation design is integral with the selection of the dam type and materials to be used. In general, dams can be classified into three categories corresponding to the principles by which their design is governed. Fill dams use large volumes of relatively low-cost, low-strength materials on typically poor foundations. Gravity dams are constructed of smaller volumes of higher cost materials on better foundations. Arch and buttress dams are constructed of still smaller volumes of material, with added costs for forming and placement, and rely on the strength of the material as arch or beam.

Spillways are designed to carry floods through the project while safely maintaining the project structures. Depending on the type of spillway, operation may or may not incur damage. Spillways can be either gated or ungated structures. There are a number of different types of spillways, each serving a unique function on a project. *Service spillways* are designed for frequent use in conveying releases of both normal flows and floodwaters from a reservoir to the downstream waterway. *Auxiliary spillways* are designed for infrequent use and may sustain limited damage when used. *Emergency spillways* are designed to provide extra protection to prevent overtopping of a dam and are intended for use under extreme conditions, such as misoperation or malfunction of a service spillway, extreme flood events, or other emergency conditions. Emergency spillways may sustain significant damage when used.

When discussing the life extension and upgrade of dams and spillways, it is easiest to classify the dams by materials used in their construction, since structures of similar materials typically experience similar problems that are remedied by similar solutions. In fact, it is more the solutions to common problems that mandate that the discussion be parceled by materials. As such, the life extension and upgrade discussion in this section focuses on three categories of materials; namely, concrete, earthen embankments, and other miscellaneous types. Concrete structures include gravity, arch, buttress, gravity arch, roller compacted, Amberson and masonry. Earthen embankment structures include those constructed of soils and rockfill materials or various combinations of the two, while timber crib dams, membrane dams, and cellular structures are representative of other miscellaneous structure types. Operational issues related to the life extension and upgrade of spillway structures are then discussed.

b) Problems

There are a variety of common problems associated with concrete dams, although most can be classified into one of three major areas:

- Stability issues.
- Leakage.
- Material deterioration.

Between 1980 and 2000, many dams within the United States have undergone extensive evaluations for these and other common problems.

Dams are traditionally designed to maintain specific factors of safety against overturning, sliding and bearing. Even if a dam survived its initial filling, the ability of that structure to meet modern day criteria or to resist changes in loading conditions attributed to increased hydrologic or seismic events, silt accumulation, or changes in the uplift profile need to be addressed.

As stated, many dams were originally designed using rule-of-thumb technology. With the advances in science and the observations of constructed dams, the engineering behind dam design and behavior has evolved into a more precise application. In addition, the criteria used to design dams has been substantially scrutinized and modified over the years. For example, it was common practice through the 1960's to consider 50% of the foundation area effective in calculating uplift loads, to use an uplift pressure at the heel of 50% of the head pond pressure, and to use a high cohesion value (through the mid 1980's) for resisting sliding. These design assumptions are no longer acceptable, meaning that many of the structures previously determined to be stable may not satisfy current design criteria.

Increases in design floods and ground accelerations during seismic events or sediment loads, can result in reduced computed safety factors, sometimes falling below acceptable values. Furthermore, the availability of sophisticated computer programs has increased the degree to which loads can be computed, and the performance of dams during extreme events can now be more accurately analyzed and predicted. As such, the extent to which one can evaluate the stability of a dam, particularly a concrete dam which tends to be isotropic and relatively easily modeled, is at a level never before seen in the history of dam design, and comes at a time when it may be needed the most, since many old designs do not meet current stability requirements.

As the primary purpose of a dam is to retain water, the passage of water through or under a dam can be a serious cause for concern. Whether this passage occurs through the pores of the concrete structure, or through joints, cracks or fissures, maintaining control of the quantity and location is fundamental to prolonging the life of a dam.

Concrete is a porous material. Therefore, some leakage is to be expected through the dam body. However, the consolidation of concrete during placement, joint preparation, mix design, or a myriad of other factors, can lead to substantial increases in leakage through the dam. Increased leakage rates lead to higher gradients, which lead to further erosion of the concrete structure and even more leakage. Leakage through the foundation is a similar problem. It can drastically change the uplift profile at the dam's foundation or within the dam body leading to a change in design assumptions. Heavy leakage through a foundation or body of the structure itself, when left unchecked or untreated, can cause major problems and even catastrophic failure of the dam.

Leakage through joints, cracks or fissures can be just as harmful. Older concrete dams that do not have modern day joint treatments and have settled due to foundation movements or temperature cycles can lose a substantial amount of water from leakage. In addition to potential losses in power generation, this leakage can lead to erosion problems at the downstream toe of the dam, as well as to changes to the uplift profiles, and significant freeze/thaw damage.

Other sources of leakage in concrete dams may be through cracks in the concrete itself, caused by tension stresses within the dam body. These tension stresses can be the result of external loads or restraint to volumetric changes within the concrete, usually associated with temperature variations at the time of placement. In addition to leakage, the cracks affect the appearance of the structure, and more importantly, the durability of the concrete.

Material deterioration due to freeze/thaw damage and Alkali-Aggregate Reaction (AAR) is a common issue related to the life extension and upgrade of concrete dams. Freeze/thaw damage results from the entrapment of moisture within a void where it freezes, expands and causes distress and progressive spalling and deterioration of the concrete. Concrete used in many older dams did not contain entrained air which helps to alleviate this problem by providing additional voids within the concrete to better accommodate the expansion. For thin structures such as slab and buttress and arch dams, extensive freeze/thaw damage can result in a reduced section, which no longer has the mass or structural integrity to resist the applied loads, leading to excessive leakage and possible structural failure.

AAR is the result of a chemical reaction between the alkali in the cement and certain silicate or carbonate minerals in the aggregate. The reaction yields a gel that absorbs moisture and swells, thus causing the concrete to swell. The rate of swelling depends on many factors, including the amount of alkalis and reactive aggregates available, the confining pressure within the structure, and temperature. The effects of AAR on water retaining structures range from minor to catastrophic. Where confining pressures are high, say at depth within a large concrete dam, the effects are minor. However, AAR within spillway piers can render a gate inoperative by slowly wedging the gate in place. Further, AAR within the upper reaches of a dam monolith can cause significant cracking resulting in adverse impacts to adjacent structures, structural distress, massive leakage and ultimately, failure of the dam.

Most major problems associated with spillways can be generally classified into four major categories:

- Insufficient spillway capacity.
- Damage to flow surfaces.
- Structural damage.
- Failure of operating equipment.

Insufficient capacity of the spillway can result from a number of different actions or physical causes. Underestimation, or a change in the peak discharge requirements of the inflow design flood, is one of the more common cases. This can be caused by a change in the hydrologic historical data applied to current conditions, changes in the requirements for computation of the peak flow or flood volume, or changes in the watershed upstream of the structure.

Flow surfaces can fail due to any number of conditions and situations. Damage by cavitation is a common occurrence in concrete spillways and many other flow surfaces in civil features of hydroelectric projects. General deterioration of surfaces due to age, freeze-thaw cycles, poor workmanship, poor materials in construction, chemicals, vandalism or many other causes is possible as discussed previously. Inadequate joint design can cause failure of the joints and materials at the joints. Abrasion of surfaces due to materials suspended in the flows, and impact with larger size rocks in areas of eddies or return flows in basins, are also common.

Structural damage is typically caused by inadequate foundation conditions, deterioration, or changes in foundation conditions or dynamic loadings. Foundation displacement may result in minor or major cracking in the structure and loss of structural stability. Seepage under or through the structure can cause loss of support and failure. Seismic loading that exceed design values can result in failure during seismic events. Inadequate subsurface investigations prior to construction or incorrect interpretation of geological conditions can lead to under-designing of key structural elements. This can result in the development of adverse foundation issues and ultimately structural instability of the spillway.

Another common problem with spillways is the failure, malfunction, or incorrect operation of equipment, causing increased flow over the spillways because design capacity of gates and/or valves is reduced.

c) Corrective Measures

Extending the life of a concrete structure can be a simple, straightforward process such as concrete repairs, which may be superficial or significant in scope. It can also be very complex and expensive, frequently requiring extensive foundation investigation programs and sophisticated, time-consuming analyses, followed by costly construction activities.

Stability issues related to concrete structures are solved by altering the state of stress within the structure through the application of external loads. This is accomplished in a number of ways. These include the addition of material, i.e. concrete; the installation of post-tensioned anchors (which has a long history of success at many projects); or the application of a jacking force within the structure to create a void that is subsequently filled with concrete which ultimately acts like a wedge.

In addition, loading on the structures can be reduced by methods such as installing drain curtains, adding additional foundation drains, or cleaning existing ones to reduce uplift loads. Piezometers can be installed to determine the actual uplift profile beneath the structure for comparison with the assumed profile. Foundation grouting, slot drains, or relief wells may be effective in reducing uplift, foundation settlements, and displacements. Seepage and uplift can be reduced by providing a cutoff wall under the structure. Using grout to provide a grout curtain, or to fill voids in the concrete, are common solutions. In some cases, a sheet pile wall can be driven to provide the necessary seepage control. Replacing or repairing the waterstops can reduce flows through and under the structure. Within certain rock types and structures, grout curtains may become less effective over time, as may curtains constructed of asphalt or other soft, non-rigid grout material.

Other load reduction techniques may involve lowering the reservoir levels adjacent to the structure, or raising the downstream water levels to better balance the hydrostatic pressures. Transferring load to more competent adjacent structures or foundation material through shear keys, buttresses, or underpinning, is another possible way of improving stability. Installing an air bubbler system on the upstream side of the structure may be an effective and low cost way to reduce ice load on a structure.

Stability improvements may also be necessary at poor quality lift joints to increase the factor of safety against sliding. The most common method of stabilization is to drill through the lift joints, and grout in high strength reinforcing bars, which essentially provides stability across the lift joint.

A clear understanding of the foundation is absolutely crucial to the stability evaluation of a concrete structure. With careful investigation and testing, foundation parameters such as cohesion, friction angles and compressive strengths can be determined with a fairly high degree of certainty. Many modern day codes recognize this and permit lower factors of safety to be used in stability analyses, potentially leading to reduced capital expenditures for stability improvements.

Attempting to reduce leakage through, or under, concrete structures can be a frustrating experience. Many times, a source is identified and mitigated, only to have another leakage source present itself. As such, proper analysis and determination of the reasons for the leakage are crucial to mitigating its effects and prolonging the life of the structure.

Leakage through cracks and joints in the structure is remedied through repair of the crack using a variety of techniques. Options include grouting, pressure injection, routing and sealing, drilling and grouting, drypack mortar, impregnation, overlays and surface treatments, flexible sealing, and stitching. The applicability of each method is dependant upon the conditions at each location, including the presence of flowing water, the condition of the concrete surrounding the crack or joint, the relative movement of the crack due to temperature or structural deflections, and accessibility. Large amounts of leakage through a concrete structure can be remedied through the installation of a new facing consisting of cast-in-place concrete, precast concrete panels, membranes, and other surface treatments.

Leakage is just one of the reasons for repairing deteriorated concrete on water retaining structures. As stated previously, if left unchecked, material deterioration can have potentially catastrophic consequences. Many of the techniques employed for sealing dam bodies are also applicable to other water retaining concrete structures such as piers, parapets, crests, and walls which suffer from cracking and spalling. Numerous specialty contractors and suppliers offer products and installations specifically designed for the repair of deteriorated concrete, as well as leaking cracks and joints. Some can even be installed under water, or against significant heads. In addition, ACI offers a number of outstanding reference guide to the causes, evaluation and repair of cracks in concrete structures, as does the USACE.

Material deterioration due to AAR, however, is remedied very differently from that resulting from erosion, cavitation or freeze/thaw cycles. As AAR is a three dimensional expansion problem, common remedial techniques involve the cutting of stress relief slots within the concrete monoliths to provide room for expansion. The size, location and orientation of the slots must be carefully selected, frequently requiring the use of sophisticated finite element models. Construction restraints and installation sequencing must be accounted for in the design of the slots, which usually mandates the involvement of specialty contractors in the design phase. Other options for dealing with AAR concrete growth involve the installation of post-tensioned anchors and shear bars, as well as the modification of adjacent structures to accommodate the growth.

On the operations side, inadequate spillway capacity can be remedied by increasing the spillway approach channel efficiency in order to increase the flow to the structure. Increasing the approach channel depth by dredging, reshaping the channel, and changing the hydraulic coefficient (by lining the channel or using other materials) can allow more flow to pass to the spillway, thus reducing potential for the accumulation or collection of floating debris which could obstruct or plug a gated spillway.

Reshaping the spillway crest, abutments, and piers can increase the discharge characteristics of the spillway, but could create additional head losses, resulting in reduced discharge capacity. Lining the spillway chute surface can also improve flow coefficients. Using hydraulic shapes with more efficient flow coefficients, and increasing and decreasing submergence, can increase flows, although this may be difficult to accomplish without affecting the flow path. In addition, lowering or lengthening the spillway crest, and increasing the head by raising the height of the dam, will also increase flow capacity through the spillway. Finally, adding additional spillway structures adjacent to, or separate from, existing spillways will provide additional capacity.

Cavitation damage to flow surfaces can be mitigated by providing aeration in the area of deterioration. Air entrainment by water surface turbulence can provide aeration, but may not provide sufficient protection for the concrete bottom of a chute. More reliable results will likely be obtained by the installation of proven aerators for chute surfaces, such as a ramp or step, with air admission by ducts, wall slots, recesses or other similar methods.

Removing the source of abrasion can reduce or eliminate abrasion of flow surfaces. Trash barriers and sediment traps can further reduce or eliminate abrasion material. Decreasing velocities and changing flow patterns can also help to reduce abrasion. A review of operating procedures and flow rate curves may identify ways to reduce high flows. Installation of a debris trap upstream of the spillway is another method of reducing the amount of abrasive material in the flow, although little can be done to prevent abrasion from suspended sediment.

Repairing flow surfaces may involve the localized damaged area or relining the entire spillway chute surface. Possible materials can be high strength concrete, epoxy, or chemically enhanced concrete. Carbon or stainless steel or other material can also be used to clad the spillway surface.

d) Opportunities

Though this section has focused on the many problems that can occur with concrete hydraulic structures, concrete as a material has many redeeming qualities when thinking about life extension and upgrade. While design of concrete structures will usually be based on its 28-day strength, concrete strength does increase over time. This strength increase can be significant when viewed over decades, resulting in increased performance. In addition, it can be a very uniform material with predictable characteristics. As such, it can be accurately modeled, with results yielding a great degree of certainty. These, along with the many other attributes of concrete, make it an ideal material for continued use.

Advances in concrete construction and concrete products, including underwater applications, has made concrete repair and rehabilitation practical and reliable. As such, extending the life of a concrete structure suffering from cracking, spalling, or other forms of material deterioration, can usually be justified when compared with the revenue generated from the continued power generation.

Concrete structures can be raised or modified with relative ease to increase the head and power production at a given project. Though the use of post-tensioned anchors, stability safety factors can be maintained when raising the height of a dam. Existing concrete can be roughened and treated to adhere with new concrete, and new concrete can be anchored to the existing concrete via drilled and grouted anchors.

4.3.3 Embankment Dams

Embankment dams are comprised of earth materials, rockfill materials or various combinations of the two. Earth dams are constructed of virtually all gradations of soil, including gravels, sands, silts, and clay, and can be homogeneous, i.e. constructed of one relatively impervious material, or may be zoned to include an impervious core protected by filter materials, drain materials and upstream and downstream shells. Improperly designed and/or constructed earth dams may also contain deleterious materials, either within the body of the dam, or along the foundation interface.

Today, most major earthfill dams are constructed of compacted soil placed in layers. The material make-up of the soil within each layer is varied in plan to form the zones within the cross section. Earthfill dams can be constructed to almost any height, and on relatively weak foundations, by flattening the slopes of the dam.

Rockfill dams, as the name suggests, are comprised primarily of rockfill. Rockfill dams constructed in the early part of the 1900's typically consisted of dumped rockfill. With the advent of modern compaction equipment in the 1950's and 1960's, the preferred construction technique moved towards placement of the rockfill in layers which are then compacted. In general, rockfill dams can be classified into two categories, the first of which are earth-core rockfill dams (ECRD) which incorporate an impervious core and filters of earthen materials within the dam cross section. The second type involves those rockfill dams which are constructed with upstream impervious linings of concrete, asphalt or geomembranes. A well known version of this type of dam is the concrete face rockfill dam (CFRD). The type of rockfill dam selected depends on various parameters which include foundation suitability, material availability, construction duration, and construction cost. Slopes of rockfill dams are generally steeper than those of earthfill dams, up to about 1.3H:1V for a CFRD, versus about 3H:1V for typical earthfill dams, because of the greater friction angle and stability of the rock fill material.

Many embankment dams have a combination of earth fill and rock fill, either in predetermined zones, or in combination. These dams must be evaluated specifically to determine their material characteristics, based on sampling and configuration. However, some older, non-engineered dams were formed using a combination of earth and rockfill, and therefore do not neatly fall into the earthfill or rockfill designations. Evaluation of these dams must be based on sampling the materials used to construct the dam in order to gauge whether its behavior is to be modeled after that of an earthfill or a rockfill dam.

A hydraulic fill dam is a special type of embankment dam formed by sluicing widely graded materials from the outer edges of the embankment towards the center. The process of sluicing deposits coarser particles towards the outer edges of the embankment, and finer particles towards the center of the embankment to form a puddle core. Although the process is an economical means for constructing a large embankment dam, controlling material density and zoning during construction is very difficult, and may lead to leakage and susceptibility to liquefaction during earthquakes. For this reason, hydraulic fill dams generally lost acceptance as a suitable dam type around 1940. However, many large hydraulic fill dams are still in service today.

While site conditions will usually dictate the most economic dam type, embankment dams, especially very large (tall or long) dams, are normally less costly to construct than are gravity dams (concrete, masonry, timber crib) because soil materials are typically less costly to use in constructing a dam, and larger mechanized equipment can be used to placed the materials quickly. Embankments dams can also be constructed on weaker foundations.

a) Function

As with concrete dams, the primary function of an embankment dam is to retain, divert or control water for the purposes of irrigation, water supply, flood control, and hydroelectric power generation. Embankment dams can be the principal water retaining structure across a stream or river to form a reservoir or maintain a specified upstream water depth, or to take the form of levees constructed along the banks of a river to control flooding. As such, they are often combined with other types of dams, such as concrete dams, for operational flexibility and overall cost savings as illustrated in Figure 4.3-1. Embankment dams are not normally suitable for use as a spillway due to the erodable nature of their materials, thus necessitating the use of a concrete spillway section.

Figure 4.3-1 Kinzua Dam (courtesy of Portland District, USACE)

In addition, embankment dams typically form the side slopes of built-up/fill sections of earthen canals used for irrigation, water supply and hydroelectric developments.

As with any type of dam, the functionality of an embankment dam depends on its ability to meet a specified set of criteria during reservoir operation and any unusual events such as earthquakes and floods. Maintaining the stability of the embankment, foundation and abutments is critical for embankment dams as with all other dam types. However, embankment dams require special care with respect to the control of seepage through the embankment, foundation and abutments. Proper attention during design and construction is necessary to assure seepage is properly addressed. Proper construction techniques including foundation preparation, moisture and density control of the fill and proper filter design/placement are crucial. Finally, diligent monitoring of the embankment dam's performance through visual observation and instrumentation is required to confirm the dam meets the design intent and is performing as anticipated.

b) Problems

Many of the common problems associated with embankment dams relate to the history of embankment dam construction and engineering. That is, prior to 1900, very little engineering went into the design and construction of embankment dams. Crude methods were used to economically achieve the end result, and for the most part, this method worked. However, as dams became larger, and their roles changed from simple, relatively low height diversion structures, to more significant water retaining structures for water supply, hydroelectric generation and flood control, concerns relative to the safety of these structures increased. Population development downstream of dams and the dramatic impacts resulting from dam failures mandated the employment of more refined engineering techniques in the design of embankment dams. In addition, experience gained through dam failures throughout the 20th century illustrated the need for improved construction techniques and quality control.

Historically, problems with embankment dams resulted from one of the following sources:

- Overtopping.
- Surface erosion.
- Deterioration (burrowing rodents, general maintenance of surfaces).
- Foundation leakage.
- Embankment leakage.
- Earthquake instability.
- Settlement.

The effects of these problems can range from minor issues that can be quickly addressed, to complete failure of the dam. As such, the corrective measures for these problems have to cover a wide spectrum of alternatives and will generally be quite site-specific in nature.

c) Corrective Measures

Overtopping of embankment dams is generally a result of insufficient spill capacity at a given dam site. The corrective measure for this situation depends on the reason for the inadequate capacity, i.e. whether it is due to an underestimation of the peak flood, revisions to the hydrological features in the catchment area, performance issues relative to the spillway, failure of a gate or other aspect of the spillway (e.g. fuse plug remaining intact), or some other factor.

Whatever the reason, additional spill capacity can resolve the issue. This can be facilitated through the installation of a new spillway structure or the expansion of an existing one. Other options include the incorporation of a fuse plug that will breach in high water conditions, modifications to the existing spill devices to increase their capacity or automate their operation, and revisions to reservoir operations that may include seasonal reservoir level reductions or changes to the flood routing procedures employed at a given site.

Overtopping protection can also be provided by raising the dam to provide additional freeboard, the vertical distance between the maximum anticipated water surface and the crest of the dam. Overtopping protection may consist of providing surface protection that would prevent erosion of the embankment materials in the event of overtopping using such means as an overlay of reinforced concrete or roller-compacted concrete (RCC), rip rap concrete blocks and rock-filled wire baskets. The construction of a parapet wall atop the crest of the dam is also an alternative to provide overtopping protection.

Surface erosion resulting from water flow can be mitigated through the installation of sufficiently sized rip rap, or the application of other protective measures such as shotcrete, reinforced concrete, roller-compacted concrete (RCC) or interlocked concrete pavers. On a grander scale, diversion structures may be installed to divert, slow or lessen the impact of the flows on the embankment dam.

It is considered a significant problem when surface erosion of relatively steep embankments occurs, particularly in homogeneous clay dams. If caught in time, replacement of the eroded area in kind is usually sufficient, but identification of the cause of the erosion is required. If an existing slope is found to be unstable during the evaluation into the cause of the erosion of the slope, additional material may be placed to flatten the overall slope of the dam or toe berms can be added to bolster the stability of the embankment slopes. Whatever the solution, identification of the cause of the surface erosion is equally, if not more important as the fix, since the cause will often dictate the viability of a given repair scheme.

A reoccurring problem with some older dams is that they incorporate what is commonly called "dispersive clays". Perry (1987) defines dispersive clays as follows:

> "Dispersive clays are a particular type of soil in which the clay fraction erodes in the presence of water by a process of deflocculation. This occurs when the interparticle forces of repulsion exceed those of attraction so that they clay particles go into suspension and, if the water is flowing such as in a crack in an earth embankment, the detached particles are carried away and piping occurs."

The inclusion of these clays does not necessarily mean that the dam is unsafe, but if not properly engineered and addressed during construction, the inclusion of dispersive clays within the dam cross section could lead to failure. In fact, some older dams which incorporated dispersive clays failed upon initial filling. Today, tests can be performed on clays to determine their relative dispersivity. Again, the fact that they are found to be dispersive does not preclude their use as embankment dam materials. It just means that this trait must be accounted for in the design and analysis of the dam.

The choice of dam type provided at a given site was often dictated by the foundation conditions. Though the foundation does not have to be impervious, seepage through the foundation must be accounted for and addressed in the design and construction of any embankment dam. Foundation treatments usually consist of installation of core trenches, cutoff walls, impervious upstream blankets, or a combination of these techniques to control the foundation seepage.

Due to the large foundation area of typical embankment dams, treatment of leaking foundations of existing embankment dams is very difficult. Complicating this issue is the fact that seepage observed exiting the downstream toe of the dam, or within the abutments, may not actually indicate the location of the foundation issue, but rather the path of least resistance followed by the water from the actual source. Seepage may occur at the toe of the dam, or hundreds or thousands of feet from the toe, depending on the foundation material. Seepage unto itself may not be of concern, depending on volume and location, but seepage accompanied by material transport (piping) is of significant and immediate concern.

The installation of upstream impervious blankets or a cutoff wall through the embankment dam may aid in reducing foundation seepage. The installation of downstream seepage berms, toe drains and relief wells may also assist in mitigating some seepage issues. Rock foundations may be treated through shallow consolidation grouting or curtain grouting.

Seepage within an embankment dam with a low permeability core is generally controlled through the design and installation of a carefully designed filter system comprised of select pervious materials. The filter zones protect the impervious core on either side and must fulfill two main functions: 1) they must prevent internal erosion of the core, and 2) they must be able to accommodate the seepage flows through the core, foundation and abutments, and safely convey the flow to the dam drainage system.

Piping, the removal of fines within a fill caused by high velocity seepage through the medium, is of particular concern in all embankment dams. If left unchecked, piping can remove enough material to potentially cause failure of the dam. The installation of filters, chimney drains, impermeable cores, downstream seepage berms, toe drains and relief wells are the most common methods of attempting to control piping and seepage effects through an existing earthen embankment.

Deformation of an embankment dam is to be expected due to settlement of the fill material over time as well as foundation consolidation. However, in concrete faced rock fill dams, known as CFRD's, formed by dumped rockfill dams, excessive deformations have lead to significant leakage separation and failure of the facing joints or cracks in facing.

Higher than anticipated deformations are often a result of poor or no compaction of the fill during construction. Remediation techniques depend upon the magnitude of the problem. For instance, deformed earthen embankments which maintain their structural integrity can simply be built up to provide the needed freeboard, though special care must be given to the determination of the core's soundness. A good portion of the early CFRD's were constructed by simply dumping the rockfill rather than compacting it, and are therefore experiencing deformations which exceed the tolerances of the facing joint details. Remedies here include the installation of a new asphalt or membrane liner over the existing facing or repair/replacement of the existing joint materials.

One of the biggest deformation related problems in embankment dams is cracking of the core due to differential settlements. This is a problem in dams where the impervious core is "brittle", usually due to compaction of the core material during construction at water contents less than optimum. This cracking is controlled by shaping the foundation to minimize the potential for differential settlements, placing the core material at proper water content and compaction effort, and proper use of filters to help seal cracks that may form in the core material.

Some older embankment dams which perform well under static loading are susceptible to severe damage and potential collapse under earthquake loading. Hydraulic fill dams, and dams constructed of relatively loose silty sands, or those located above foundations comprised of similar saturated, cohesionless materials, may be susceptible to liquefaction under cyclic shear loading imposed by earthquake motions. For example, the 1971 failures of the 142-ft lower and 82-ft upper San Fernando Dams near Los Angeles, CA are attributed to liquefaction of these hydraulic fill dams as a result of a magnitude 6.6 earthquake.

Liquefaction evaluation of dams and their foundations has become common practice since the 1990's, and should be performed for any dams constructed of, or on, potentially liquefiable materials. Should the materials in the dam or foundation be found to be susceptible to liquefaction, several measures are available for stabilization of the material. The installation of stone columns, soil-mixing, grouting and other foundation densification or modification techniques have been successfully used to treat dam foundations. However, in some cases, adequate treatment of the dam or its foundation may not be possible and it may be necessary to construct a new dam downstream of the existing dam.

d) Opportunities

The need for the rehabilitation of an embankment dam is often facilitated by safety concerns. That is, a deficiency is either noted through visual inspection of the dam, or identified in the regular analytical review of the dam's design and the hydrological aspects of the project's drainage basin area. When a deficiency is noted and work is required, opportunities to add value to the project may be available. For instance, if a hydrological review has revealed that the design peak inflow into a given reservoir will raise the design water level to a height which would overtop the embankment dam, and a decision has been made to raise the embankment dam to provide overtopping protection, it may be worthwhile to investigate the incremental cost of adding sufficient discharge capacity and permanently raising the reservoir to provide additional head for associated hydroelectric facilities.

An inherent opportunity exists when mitigating seepage through any embankment dam. Reducing the seepage through the dam bolsters its stability and overall safety, while at the same time providing additional water for any hydroelectric generation at the site, associated irrigation, or water supply.

4.3.4 Timber Crib Dams

a) Function

The use of a timber crib filled with ballast is an outdated means of constructing low head dams. Typically, timber crib structures were constructed of wood cells pinned together and filled with rock or other ballast. They were a common means to construct a dam from the 1800's through the 1940's. The dams were commonly constructed in heavily forested areas, and were popular due to the fact that they did not require specialized skills to build.

The timber crib structures relied on the weight of the ballast to provide the required stability and sliding resistance, and can be founded on both earth and rock foundation materials. The rock fill was considered to be a free draining material, and uplift pressures were not assumed to develop beneath the dam. Upstream and downstream faces would be typically covered with wood planking to retain the headpond and to protect the otherwise exposed timber cells from damage under overtopping conditions.

b) Problems

Typically, aging causes deterioration of the timbers and results in increased leakage and sagging of the crest, especially in the upper ten feet of the dam that is exposed to wet and dry cycles. In general, the wood structures provide a life generally defined as 60-75 years, with maintenance depending on the type of wood used in the construction and the geographical location of the dam.

Timber crib structures are highly susceptible to damage under flood conditions. Unfortunately, repairs are difficult due to the need to "disassemble" the structure to gain access to the areas needing repair. When the planking on the upstream and downstream sides have been removed, have deteriorated, or have been damaged to the point where they are missing, the structure is most susceptible to damage from any overtopping condition.

Leakage through the structure can be high, depending on its condition, even when repaired. For example, a 60-year old, 300-foot long by 25-foot high timber crib dam had measured leakage in excess of 150-cfs prior to rebuilding, with 15-cfs of leakage after rebuilding the crest and re-planking of the upstream face.

c) Corrective Measures

In short, nothing substantial can be done to prevent deterioration of the timbers. Timber crib structures will have to be rebuilt periodically. Excessive leakage not only impacts on generation, but can result in the erosion of the wood planking and timbers, as well as earth and soft rock foundation materials.

Leakage can be minimized by maintaining the upstream planking in a sound condition. A limited amount of leakage is needed to preserve the internal timbers by keeping them constantly wet, rather than subjecting them to wet-dry cycles. Leakage most often occurs in the top ten feet of the dam's height, particularly along the joints in the planking. Maximum leakage resistance is provided by double planking the upstream face, with the top layer covering the joints in the layer below (ship lap).

To further seal the joints maintain a layer of clay or sediments on the upstream face. Note, however, that a waterproof membrane or sheeting of any kind must not be installed on the upstream face of the structure, since the elimination of all leakage through the wood planking will actually increase the rate of deterioration of the wood by allowing the wood to dry out.

Any rebuilding of timber crib structures should be done using planking and timbers of equal or greater size, and using steel pins and spikes also of equal or greater size. Timber cribs can be encased in concrete if care is taken to allow the draining of the wood cribbing. Concrete capped timber cribs are susceptible to loss of the downstream face if the dam is not adequately drained, or if the concrete is not adequately interlocked with the timber cribbing.

d) Opportunities

Opportunities for improving the performance of a timber crib dam are few, and limited to modifications as needed to maintain or, possibly, extend the life of the structure. Deterioration of the timber materials and damage by floods are the primary causes for the loss or failure of a timber crib structure. The design, construction, and repair of timber crib dams is quickly becoming a lost art and few, if any, new timber crib dams are being built.

4.3.5 Reservoirs

Inherent with the construction of a dam is the creation of a reservoir, the term used to describe the body of water impounded behind the dam. Though not a structure in itself, a reservoir plays a significant role in the design and operation of a hydroelectric project. As such, a brief discussion is included with this section. Reservoir, however, is really an all-inclusive term. On long canal projects, the reservoir is the water within the canal and forebay whereas on run-of-river projects, it is the river leg immediately upstream of the dam.

a) Problems

Aging reservoirs usually contain significant amounts of sediment, debris and other materials that can adversely affect the flood discharge capability of a downstream dam. This material can accumulate to an extent that it becomes a threat to good operation of the dam, low level outlets, dam inlet gates and penstocks.

Increasing or decreasing the capacity of the reservoir may be required to extend the life of the project. Increasing the capacity may be for additional flood storage or increasing the volume that may be delivered to the hydroelectric unit. Decreasing the reservoir level may be done to reduce the load on the dam, which may increase the stability or reduce seepage.

Significant changes in the reservoir elevation may have equally significant impacts on the water quality. Increasing the reservoir volume may decrease the concentration of naturally or artificially occurring contaminants. These costs, and political impacts of lowering or raising the reservoir elevations, will need to be carefully studied before lowering the reservoir.

b) Corrective Measures

Regarding sedimentation, routine bathometric measurements of the reservoir bottom surface can be used to provide quantification, frequency and locations of deposits. Divers are often used to do more precise investigations around inlets and gate structures.

Sediments that will be dredged or sluiced downstream will need to be evaluated to determine if they contain contaminants that will have negative impacts on the water quality for both fisheries and domestic water supplies. Gate structures and valves that are used for sluicing should be examined and a determination made to estimate the wear effects on these gates or valves. Sediment sampling for grain size distribution and roughness will be necessary if removal by mechanical dredging or hydro-suction is to be considered.

Larger debris removal is usually performed by cranes with clams and buckets, large backhoes and trucks. Bathometric measurements are used to determine the quantity of the debris. Divers are often used to verify the electronic measurements and general size of the debris, and are used to assist in attaching cables or ropes to large debris items for removal by cranes.

There are many ways to economically increase the height of the dam and increase the reservoir storage. Often, the spillway needs to be modified to permit a greater height while still maintaining the design capacity. The addition of a labyrinth or fuse plug spillway will allow a higher normal pond elevation while maintaining adequate flood discharge capacity.

Decreasing the reservoir elevation is often easily achieved with little or no modification to the other project facilities. The exception may be when the reservoir elevation has historically been held constant for a very long period of time. Lowering the reservoir in these cases usually dramatically impacts recreation amenities such as boat ramps boat docks, swim beaches and any many other related facilities. Lowering the reservoir can also result in increased amounts of sediment and debris transport within the reservoir due to the exposing of "stable" materials to higher erosive velocities. If the rate of reservoir drawdown is excessive, stability of embankments around the reservoir can be adversely affected.

Raising and lowering the reservoir may dramatically impact the wetlands and other habitat in the upper and usually shallow areas of the reservoir. Lowering water levels may drain shallow wetland areas that provide specially habitats. Flooding these wetlands, by raising the reservoir, will also change wetland ecological system. The new higher water surface will, with time, generate new wetland areas. Wetlands and other similar regions are usually federally protected and require significant permitting activities.

4.3.6 Case Histories

a) Concrete Dams and Spillways

No. 1 Spillway Surface Deterioration
Inghams Hydroelectric Development (Brookfield Power New York, 1997)

The Inghams Hydroelectric Development is a 7 MW facility located on the East Canada Creek in the Town of Manheim, N.Y. Its average annual generation output is about 28,700 MWh. The dam is a concrete monolithic structure, built in 1912, that is presently classified as high hazard. It was stabilized in 1995 using tendon anchors. During the past few decades, some concrete repair work was performed on the 205 ft. long by 25 ft. high spillway, but no major rehabilitation was done.

In 1997, the condition of the spillway was considered poor (Figure 4.3-2). Its defects included: cracked concrete, deep gouges in the surface especially near joints, concrete spalling as deep as 12 inches, undermining of the toe, and 54-inch high wooden flashboards that were unreliable and expensive to replace on average twice per year. These were the main reason for significant upstream pond fluctuations and resident complaints.

Figure 4.3-2 Inghams Spillway Deterioration (courtesy of Brookfield Power New York)

Major rehabilitation of the spillway was performed in 1997, and included removal and replacement of a minimum 8 inch thickness of reinforced concrete from both abutments, the spillway crest, spill surface, toe, and portions of the upstream face; reconstruction of the undermined areas of the spillway toe; installation of a new 54-inch high inflatable crest control system (rubber dam) with local controls, and erection of a fall protection cable to which personnel could attach for safety when accessing the spillway crest (Figure 4.3-3).

Figure 4.3-3 Inghams Rebuilt Spillway (courtesy of Brookfield Power New York)

The new crest control system also permitted significantly better and quicker control of pond levels, thus reducing upstream flooding and the significant 5-foot fluctuations of the pond that sometimes disrupted recreation. In addition, there was an estimated 890 MWh per year increase in hydroelectric generation due to the ability to quickly raise the crest control device after a flood, thereby maintaining a more consistent pond level throughout the year.

No. 2 Dam Instability Due to Flood Loading Niagara Hydroelectric Project (AEP, 1997)

The Niagara Hydroelectric Project was constructed in 1906 and is located on the Roanoke River in Virginia. It is a concrete gravity dam with a height of 50 feet and an ungated spillway length of 452 feet. It has two generating units producing 2.4 MW of total capacity.

The spillway for the project had inadequate factors of safety for the inflow design flood loading condition, and required stabilizing. The spillway was constructed out of cyclopean concrete.
Two options were considered for remediation of the spillway. The first option was to buttress the downstream face with grouted riprap, and the second option was to use roller-compacted concrete (RCC). The second option was selected using RCC with a two-foot sacrificial outer surface and a conventional concrete cap. The RCC option was selected because regulating agencies had concerns regarding quality control in

constructing, and longevity of the grouted riprap option. Figures 4.3-4a and 4.3-4b show the Niagara Project before and after the remediation.

Figure 4.3-4a Niagara Before Remediation (courtesy of American Electric Power)

Figure 4.3-4b Niagara After Remediation (courtesy of American Electric Power)

No. 3 Dam Instability Due to Ice Loading
Twin Branch Hydroelectric Project (AEP, 1999)

The Twin Branch Hydroelectric Project is located in northern Indiana on the St. Joseph River. It was constructed in 1922 and is a timber crib dam with a height of 37 feet. The 402-foot spillway consists of 198 feet of gated spillway, with 7 tainter gates and 204 feet of flashboard spillway. Power production consists of eight Flygt units with a total capacity of 4.8 MW.

The spillway at the project had inadequate factors of safety for overturning due to ice loading and required remediation.

To resolve the issue, an air bubbler was installed below the water surface in the impoundment on the upstream side of the dam, which was operated during the winter months to prevent ice from forming against the upstream side of the spillway. The system consisted of a large compressor, located in a building on the intake deck, which feeds compressed air to a pipe with nozzles located upstream of the spillway, creating bubbles which keep the water from freezing. One problem with the system is that if it is not operated every few weeks, year round, the nozzles plug up with zebra mussels and divers need to be brought in to clean the nozzles.

No. 4 Insufficient Spillway Capacity
Brule Dam (MWH, 1988)

In 1988, Wisconsin Electric evaluated the spillway capacity expansion needs of the Brule Project. Work included PMF Power Company determination for several projects and development of a complex dambreak/floodwave model. The model was used to assess incremental stage rises and associated damages, as well as to identify the appropriate inflow design flood for each project in the system.

The dambreak/floodwave work was then used as the basis for evaluating the practicality of developing nonstructural early warning plans in lieu of structural expansion of existing spillways.

Figure 4.3-5 Brule Dam (courtesy of MWH)

Ultimately, Brule Dam, a 60-foot-high combination earthfill and concrete dam constructed in 1916, was identified as a priority project for corrective action. The 50% PMF was selected as the appropriate inflow design flood. Design work was initiated and completed for rehabilitation of the existing tainter gate controlled spillway, including concrete resurfacing at the Brule Dam. In addition, a new, two-stage, fuse-plug controlled, side channel auxiliary spillway was designed and constructed in rock around the left abutment of the main earth embankment to solve the spillway capacity problems.

No. 5 Inadequate Seismic Stability & Spillway Capacity Devil's Gate Dam (MWH, 1997)

Devil's Gate Dam, completed in 1920, is a 103-ft high concrete-gravity arch dam located near Pasadena, California. Its primary purpose is flood and debris control. Owned by the Los Angeles County Department of Public Works, prior to rehabilitation, the reservoir had been restricted by the State of California, Division of Safety of Dams (DSOD), due to stability concerns during a seismic event, and because of inadequate spillway capacity to pass the runoff resulting from the Probable Maximum Precipitation (PMP) event.

Figure 4.3-6 Devil's Gate Dam (courtesy of MWH)

Five spillway expansion alternatives were prepared, with the final spillway capacity increase achieved by replacing the existing ungated spillway with an orifice and overflow spillway headworks and a new reinforced concrete spillway chute.

Final designs had to be developed to bring the dam into compliance with State of California, Division of Safety of Dams (DSOD) criteria. Geotechnical analyses were performed including rock, soil drilling as well as testing, and installation of survey monuments and piezometers. The results of the investigations were used in stability analyses and hydrogeologic analyses of the dam and spillway structures under normal and seismic loadings.

Seismic stability analysis of the proposed spillway headworks included a liquefaction analysis of the foundation soils and post-earthquake stability analyses. Headworks stability required a 75-foot thick soil-cement anchored foundation to provide sliding stability against peak ground accelerations to strengthen the existing dam. Recommended measures to increase dam stability included a 10,000-cu.yd. roller compacted concrete (RCC) buttress, foundation drains, and an upstream seepage cutoff grout curtain.

Spillway excavation incorporated pattern rock bolting and shotcreting to prevent undermining of the existing bridge piers. Flow was channeled into the spillway by a new mechanically stabilized earth (MSE) retaining wall. Additional features designed for the project included a 60-foot long bridge over the spillway, an earth dike on the right abutment, and habitat restoration alternatives for a 2,500-foot segment of the Lower Arroyo River downstream of the dam and a recreation area upstream of the dam.

The RCC buttress was modified during construction to utilize conventional unreinforced concrete due to the small amount of material and restricted access to the placement site. The change to conventional concrete did not increase the project cost. The project was completed in 1997.

No. 6 Erosion of Unlined Discharge Channel
L.L. Anderson Dam (MWH, 1999)

Placer County Water Agency (PCWA) owns and operates the Middle Fork American River Project (MFARP), FERC Project No. 2079. The 231-ft high L.L. Anderson Dam (also known as French Meadows Dam) forms the 136,000 acre-feet French Meadows Reservoir. The MFARP was constructed during 1963-1966, and formal operation was commenced in October 1966.

In May 1996, record snowmelt runoff caused a peak outflow of approximately 6,000-cfs through the L.L. Anderson Dam spillway, causing severe erosion in the lower portion of the unlined discharge channel. The spillway discharge breached the channel, eroding weathered bedrock and overburden along a pre-existing shear zone in the bedrock that intersected the channel. Eroded debris temporarily dammed the Middle Fork of the American River causing the backed-up water to partially flood the outlet works ring-jet valve and the toe of the dam.

Spillway modifications were designed to reduce the potential for future erosion and to assure the ability to operate the ring-jet valve in the event of the Probable Maximum Flood (PMF) outflow of 15,500 cfs. The project included lowering and widening the spillway channel; creation of a series of hydraulic steps up to 20 feet high to dissipate energy; construction of a plunge pool and weir to further lower the energy and redirect flows, construction of rock-reinforced and shotcrete-lined channel walls to control erosion and direct the flows away from the dam and outlet works; and reorientation of the end of the channel to improve flow conditions into the river. Construction began in June 1998 and was completed in November 1999.

No. 7 Concrete Deterioration (Alkai-Aggregate Reactivity)
Lake Decatur Dam (MWH, 2001)

Lake Decatur Dam, owned by the City of Decatur IL, is a 30ft (9gm) high dam, comprised of two 600ft (180m) -long embankments, a 625ft (190m) long bascule gated concrete spillway and a concrete outlet works with two 9ft (2.7m) x 13ft (4m) sluice gates. The dam forms a reservoir that supplies water to the City of Decatur, as well as heavy industrial manufacturing plants in the area.

In November 1999 and again in December 2000, dam safety inspections in accordance with the requirements of the Illinois Department of Natural Resources, Office of Water Resources were conducted. These inspections revealed several areas of severely deteriorated concrete. This deteriorated concrete exhibited signs of potential alkali-aggregate reactivity. Investigative studies to determine the cause of the severe concrete deterioration were conducted. Much of this deterioration was located on a spillway access platform, originally constructed over a fish ladder. The fish ladder was abandoned and backfilled with concrete. An overlay was placed over much of the downstream face of this access platform. Three cores were taken in the area that exhibited the most severe deterioration at the dam. Petrographic analyses were performed on the samples. Tests revealed reactive aggregates in the original concrete, the overlay concrete, and the original overlay concrete interface.

As a result of this analysis, drawings and technical specifications were prepared to address the reactive aggregate in the overlay and access platform. Expansion joints were placed along the entire length of the gated spillway overlay to allow for aggregate expansion. Alkali-aggregate reaction is accelerated with the intrusion of water, and therefore, epoxy was injected into cracks along the upstream face of the access platform to tighten the face and prevent water from penetrating to the downstream face of the platform. This work was scheduled in the winter months, when thermal shrinkage of the concrete maximized crack widths. Since Lake Decatur provides water supply for the Decatur, Illinois, the reservoir cannot be drawn down at any time. Therefore, provisions and special underwater details were employed in the contract documents to perform all modifications underwater.

No. 8 Instability (High Uplift Pressures)
Harlan County Dam (USACE, 1975)

Harlan County Dam, owned and operated by the United State Army Corps of Engineers, Kansas City District, is located on the Republican River, 240 mi (384 km) above the mouth in south central Nebraska near the town of Alma. The primary purposes of the project are flood control and irrigation, with incidental benefits for recreation. At full pool, (elev. 1973.5) the lake has a capacity of 850,000 acre-feet, a surface area of 22,800 acres and extends 17 mi upstream. The dam is a rolled earth fill embankment with a centrally located gravity overflow spillway. The embankment is approximately 12,000 ft long. It has a top width of 30 ft, contains 13,400,000 cu yd of fill, and extends 107 ft above the streambed.

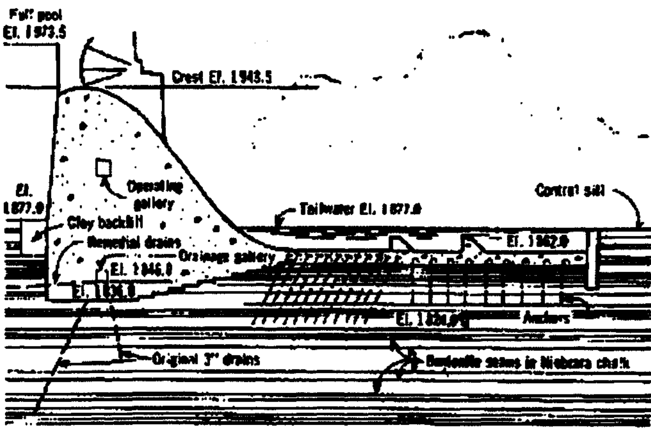


Fig. 1. The gravity concrete overfall spillway at Harlan County Dam, Neb., is 856 ft (260.9 m) between training walls and has 18, 40 by 30-ft (9.1 by 12.2-m), tainter gates for releasing floodflows. Nine, 5 by 8-ft (1.5 by 2.4-m), sluices are used for normal releases. To assure stability of this spillway, horizontal drains were placed to relieve hydrostatic pressures on the dam face

Figure 4.3-7 Harlan Dam (courtesy of Portland District, USACE)

Prior to, and during the construction of Harlan County Dam, an extensive exploration and testing program was carried out. The testing was done on a thick bentonite seam after construction had commenced. During construction the spillway was instrumented to monitor performance of the structure. The stability of the spillway was based on the condition of the structure sliding on the bentonite seam. Resistance was provided by a horizontal column of assumed sound chalk tied together with anchor bars and by a passive wedge at the downstream end of the column. Anchors were installed in the stilling basin for additional resistance to sliding.

As part of a reevaluation program, the stilling basin was dewatered and a thorough inspection made of the basin and control sill. No visible signs of excessive movement or distress were observed during the inspection. Data from the instrumentation were studied in detail during the reevaluation. It was concluded that if the structure had undergone deep-seated movement, it would not have been detected with the existing instrumentation. The existing, nearly vertical foundation drains were very effective in reducing pressures under the structure, but the hydrostatic pressures at the upstream face of the structure and in the foundation upstream of the structure were essentially the same as the reservoir pressure.

The studies clearly indicate that the uplift assumptions on bentonite seams beneath the structure, including the stilling basin, have very little effect on the structure's stability, due to the low friction of the bentonite. Uplift assumptions on the passive wedge, however, have a much larger effect because of the higher friction value assumed for the pre-sheared chalk surfaces. All analyses assumed a horizontal driving force at the upstream face of the structure equal to full hydrostatic based on full pool. Newly installed piezometers substantiated the existence of nearly full reservoir pressure against the upstream face of the structure. They also indicated that the nearly vertical existing drains under the structure were almost 100% effective in controlling uplift. The upstream horizontal pilot drains effectively lowered the hydrostatic pressure both at the structure's face and in the chalk foundation. Changes in the piezometric pressures during the drilling for these devices indicated there is hydrostatic communication along the upstream face for distances of up to 300 ft horizontally.

Eventually, the alternates were reduced to a plan for installing additional foundation anchors in the stilling basin area or installing an upstream drainage plan consisting of 200-ft long, nearly horizontal 3" drains from the drainage gallery at 10-ft spacing throughout the length of the spillway. The drainage plan was selected because the cost of the anchor plan was estimated to be $370,000, compared to $80,000 for the upstream drains. The drains were drilled from the drainage gallery. Therefore, the drilling required specialized equipment and methods due to the space limitations of the 5 by 7-ft (1.5 x 2.I-m) gallery.

The final operation was the drilling of 87 drain holes through the upstream wall of the gallery. With installation of the remedial horizontal drains, the hydrostatic levels at both the upstream face of the structure, and in the chalk foundation, have been substantially reduced. The reevaluation studies, under full pool conditions based upon conservative foundation shear strengths, and with the remedial drainage system functioning, indicate satisfactory safety factors.

No. 9 Insufficient Spillway Capacity and Stability
Wahleach Hydroelectric Project (Hartford, 1998)

The 60-MW Wahleach hydroelectric facility, completed in 1952, is located approximately 75 miles (120 km) east of Vancouver. It consists of Wahleach Dam, Boulder Creek diversion dam, which diverts Boulder Creek into Wahleach Lake, a power intake structure on the west side of the lake leading to a tunnel , a short-surface penstock leading to a powerhouse containing one turbine-generator unit under 600m of head.

The dam is a zoned earthfill structure, 70ft (21m) high with a crest length of 1370ft (418m), constructed on overburden comprising layered glacial deposits. A free-crest overflow spillway, located on the right abutment, discharges into an excavated channel, which joins the original Wahleach Creek some .5 miles (800m) downstream of the dam. Boulder Creek, which accounts for about one third of the inflow into Wahleach Lake, was diverted at the time of construction along a 400m long excavated channel to discharge into the reservoir just upstream of the spillway.

The dam safety issues at Wahleach basically began in 1989, when the Wahleach Dam was classified as a high hazard facility. The specific dam safety issue evaluated with risk assessment included flood and the potential for flows to erode material downstream of the spillway. BC Hydro conducted a risk assessment by: (1) identifying events that could lead to failure, (2) assigning probability of these events,

(3) calculating failure probability resulting from these events, (4) estimating consequences of failure and calculation of risk.

The risk assessment team carried out a failure pathway analysis by obtaining details of all possible dam failure mechanisms that floods could induce. Considering the two potential failure pathways, Erosion Path 1 represents dam failure and reservoir release by a succession of erosion-induced failures of elements along the center line of the spillway; that is, failure starting with the downstream channel and eroding back under the concrete stilling basin, spillway chute, concrete overflow weir, and finally eroding the approach channel. Erosion Path 2 represents erosion of the right end of the dam where it abuts the concrete spillway. The erosion would occur from the downstream side of the dam by return currents from the spillway discharge. Since the 60 MW facility represented a significant capital investment, BC Hydro considered alternatives to modification of the spillway to extend the service life of the project.

Given the results of the risk analysis, BC Hydro then made both physical improvements and operational changes that increased the facility's reliability and performance. The overflow ogee weir was replaced by a new structure of the same hydraulic shape as the original weir. Hydraulic model tests showed that the weir can pass PMF discharges and that the weir experiences positive pressures for flows up to and including the half-PMF. Under PMF flows, the tests indicate slight negative pressures on the upper quadrant of the crest, but positive pressures are indicated on the downstream portion. The weir is considered adequate for all flows.

The spillway chute was constructed such that there are no significant flow concentrations. The water depths along the walls are low, and nearly uniform flow exists across the chute as the flow enters the stilling basin. The walls of the chute and stilling basin were constructed to contain PMF flows. The invert level of the stilling basin was based on PMF peak flow such that the hydraulic jump would remain within the stilling basin, as long as excessive scour does not occur downstream and cause a low tailwater level. The 1.0-m-high baffle blocks are expected to be effective for flows up to the PMF. Hydraulic model tests indicate that the stilling basin can safely pass PMF discharges provided that tailwater control is maintained.

The downstream discharge channel was protected with a double layer of rip-rap for a distance of 300ft (90m) downstream of the stilling basin. The protection consists of 66ft (20m) of D50 = 3.3ft (1.0m) rip-rap immediately downstream of the basin and a further 230ft (70m) of D50 = 2.3ft (0.7m) rip-rap. The rip-rap was designed to contain half-PMF flows without any channel degradation. It is anticipated that half-PMF flows will degrade the channel downstream of the rip-rap, but such degradation, while displacing the placed material downwards, has a low probability of eroding back to the stilling basin due to the protection afforded by the rip-rap.

A sheet pile cut-off was provided at the downstream end of the stilling basin to safeguard against erosion and undermining of the foundation materials beneath the stilling basin. The sheet pile cut-off, which is about 20ft (6m) deep, spans almost the entire width of the stilling basin at the downstream end, wraps around, and continues upstream along the outline of the stilling basin on both sides for a distance of about 30ft (8.8m). A spillway foundation drainage blanket was built.

After implementing the physical improvements, a potential weakness in the dam's ability to safely discharge floods that exceed the inflow design flood remained. Specifically, when floods exceed the inflow design flood (half-PMF), erosion of the downstream channel could undermine the spillway and cause the dam to fail. However, before floods approaching half of the PMF occurred, the community of Laidlaw would flood due to the overbank flow from the creeks adjacent to Wahleach Creek.

An Early Notification System (ENS) has been established to improve the security of the residents of Laidlaw from all floods. The ENS comprises five elements: detection, evaluation, notification, warning, and evacuation. Appropriate actions are taken based on criteria set out in an action initiation matrix, which stipulates action to be taken for observed conditions as well as for forecast hydrological information and potential flood conditions.

This example demonstrates a life extension and upgrade plan by an owner, when spillway capacity requires reevaluation due to changes in computational methods for PMF. The overall result was a significant increase in public safety, and long- term life- extension of the facility.

No. 10 Concrete Deterioration and Insufficient Dam Stability Rainbow Dam (Kleinschmidt Associates, 1992)

The Rainbow Hydroelectric Project is located in Windsor, CT. The project includes a 400-foot long concrete gravity spillway with six-foot high wood flashboards. The dam has a structural height of 58 feet, to develop the site's 60 feet of gross head for two 4 MW turbines. The Project was constructed in 1925 to provide electric power to a local industrial manufacturing plant. The Project is not regulated by the FERC but is subject to Dam Safety Regulations of the State of Connecticut.

Monitoring of the spillway dam had determined that the structure had lost an average of six to eight inches of concrete from the surface of the dam (see Figure 4.3-8). While sub-freezing conditions are common in Connecticut, the deterioration of the concrete surface was exacerbated by the nearly continuous leakage that flowed over the surface due to the condition of the wood flashboards. Because the river is heavily regulated, the flashboards are not released or replaced on any frequency, which results in a continuing increase in leakage. In addition, the soft surface of the concrete had allowed vegetation to grow, further increasing the freeze-thaw damage by promoting penetration of water into the concrete surface, and promoting erosion of the concrete surface. Again, because the river is highly regulated, there is infrequent and insufficient spillage over the flashboards in which to wash away the vegetation.

Figure 4.3-8 Rainbow Dam (courtesy of Kleinschmidt Associates)

To restore the condition of the spillway, the downstream surface was resurfaced with 36-inches of cast-in-place reinforced concrete (see Figure 4.3-9). The concrete was a 4,000-psi mix, with an air entraining admix to provide protection against freeze-thaw damage, and superplastercized to reduce the water to cement ratio. The life expectancy of a resurfacing overlay should be at least 50 years, and probably 75 years if appropriate measures are taken to address surface preparation, leakage, and freeze-thaw. While resurfacing overlays are normally 8 to 12 inches thick, the Owner elected to provide a heavier overlay at the Rainbow Dam to increase the stability of the spillway to meet the dam safety standards of the State of Connecticut. The cost of the stabilization was the incremental cost of the additional concrete needed (approximately 1,625 cy for the additional 24-inches of thickness).

Figure 4.3-9 Rainbow Dam, Repairs (courtesy of Kleinschmidt Associates)

The concrete overlay was mechanically fastened to the existing dam by the grouting of reinforcing dowels into the existing dam. Minimal grouting of leakage through the existing dam was performed; rather, the leakage was collected and piped through the concrete overlay. It was determined that it was better to let the minimal leakage continue from a known location rather than attempt to grout the leak, only to have it reappear in another location some days, or months, later. During placement of the concrete, the piping system allowed leakage to be passed through the fresh concrete without washing of cementious materials.

The owner also satisfied the state's environmental agencies by adding downstream fish passage facilities to bolster the upstream fish passage that had been installed in the 1970's. The downstream fish passage facilities included sorting and holding facilities for scientific-biological study and ultimately increased the cost of the project by 45%.

No. 11 Soluble Soils & Embankment Dam Failure
Quail Creek Dam, (UT DWR, 2002)

Quail Creek Reservoir is a large impoundment of Quail Creek and several tributaries in extreme southwestern Utah. This reservoir provides drinking water to St. George and offers many recreational opportunities. At 12:08 a.m. on January 1, 1989, the southwest dike failed and unleashed 25,000 acre-feet of water forcing the evacuation of downstream residents, and causing fairly extensive property damage. The dike had been plagued by leakage since its completion in 1985, but was considered safe until shortly before the flows began to increase prior to its collapse. The leakage was due in large part to the solubility of gypsum found in the soil, which dissolved and produced conduits for the transmission of water in the area.

Several attempts were made to reduce and control the flow past the dike but new leaks continued to appear. The leakage increased significantly prior to the failure. Work crews battled for 14 hours to seal the leak in the earthen dike including efforts to pump concrete grouting into the toe of the dike. Despite efforts the situation got progressively worse. By 10:00 p.m. on New Years Eve, it was apparent that failure of the dike was imminent. Although New Years Eve celebrations caused some difficulties, efforts to warn residents and conduct needed evacuations worked well and ran smoothly, which prevented injury and the loss of life.

After the failure of the original earthfill dike in 1989, the dam was reconstructed as a roller compacted concrete (RCC), structure with a concrete and RCC cutoff trench which reached a depth of about 75 feet (22.9 m) through the maximum section, a maximum dam height of about 80 feet (24.4 m), and a crest length of about 2150 feet (655 m). Currently, water is used for both municipal/industrial purposes and irrigation, but as growth in the St. George area continues without water conservation measures, all of the water will be required for municipal/industrial purposes.

Since completion of the new dam in 1991, seepage has gradually increased. Seepage had been most notable along the left side of the dam, leading to the installation of a toe drain system. During the past few years, subsidence features have been noted down stream of this area. Since January 2002, seepage along the right side of the dam has increased significantly. A series of exploratory holes are currently being drilled in target areas, along with research into various geophysical methods, which may assist in finding the locations of seepage beneath the cutoff trench so that an appropriate grouting plan or other remediation procedures can begin.

No. 12 Alkali Aggregate Reactivity
Fontana Dam (Meisenheimer & Wagner, 1997)

The Fontana Dam is a 480-ft high concrete gravity dam that was constructed by the TVA during the period of 1942 to 1945. In 1972, a large crack developed at the curved portion of the dam. Measures taken at the time included lowering the reservoir and strengthening the cracked blocks with post-tensioned anchors. In 1976, a four-inch wide slot was cut to a depth of 100-ft using overlapping drill holes. The slot had closed at the top by 1984 and the top third was then recut to a width of five inches. Post-tensioned anchors were also installed across the top of the dam in 1984 to improve the seismic stability in the upper portion of the dam. Horizontal cracking along construction lift joints, also the result of AAR growth, was responsible for the instability.

TVA had performed monitoring of the dam since its construction. After the crack developed in 1972, extensive instrumentation was installed and monitoring was performed before and during slot cutting. Exploration and in-situ overcoring stress measurements were obtained at different periods. The slot closure and stresses below the slot have been monitored continuously since slot installation.

The main spillway and emergency spillway have AAR related problems as well. The gates at the main spillway have experienced binding and have been trimmed several times. Very little additional trimming can be performed on these gates. The emergency spillway has deflected approximately 18 inches upstream at the center and joints have opened, causing concern about the arch action at the top of the spillway.

Slots will be installed to address the AAR growth for the next two decades. A 3D finite element model has been developed and calibrated for the dam, along with a detailed submodel for curved section. The models are used to determine the present state of stress and currently, the preliminary analysis indicates that no slots are required at any locations other than the dam's curved portion.

Various stress relief slots are being evaluated to relieve the main spillway gate binding problems and to determine whether the existing slot needs to be recut. Finite element models are being used to determine the optimal slot locations, number, depth, cut width, and installation sequence.

No. 13 Alkali Aggregate Reactivity
Hiwassee Dam (Meisenheimer & Wagner, 1997)

The Hiwassee Dam is a 307-ft high concrete gravity dam that was constructed by the TVA during the period of 1936 to 1940. The main spillway is located in the center of the dam and has seven radial gate bays.

The dam has been experiencing AAR growth and shows no signs that the growth rate is slowing. High stresses and deflections within the dam have resulted in significant cracking in the upper portion of both the right and left non-overflow sections. Numerous cracks have been mapped in the drainage gallery, on the spillway training walls, in the switchyard wall and foundations, and in the gate hoist chamber. Spalling has occurred at construction joints, the bridge, and other areas of the dam. The non-overflow blocks have deflected into the spillway opening causing binding of the outer radial gates. These gates have been trimmed as much as possible without a major rehabilitation of the gates.

In 1990, TVA initiated a proactive program for the Hiwassee Dam to develop a long range AAR management program. Finite element models were developed to define the state of stress within the structure, define appropriate remedial measures that would provide 25 years of AAR management control, and be used as a predictive tool for future AAR growth. Extensive instrumentation and monitoring was implemented to obtain data that would be beneficial for calibration of the finite element models and to define some of the stress temperature conditions that affect concrete growth rates, both prior to and after remedial construction.

The models and submodels were used to determine the optimum location, depth, and width for slots on either side of the spillway to relieve the radial gate-binding problem and to provide overall relief for a 25-year period. Post-tensioning requirements to prevent shearing during slot cutting was also defined by the models as were the locations of slots to isolate the curved non-overflow sections at each abutment from the main portion of the dam. Permanent blister cofferdams were installed on the upstream face of the dam so that future slot cutting could be performed without requiring reservoir drawdown.

The models were found to be very accurate, both in terms of predicting when the slots would close as well as the rebound of the spillway. The use of slots on either side of the main spillway resulted in a considerable cost savings by eliminating gate binding without performing an expensive gate modification.

No. 14 Foundation Leakage
'El Cajon' Francisco Morazan Project , Honduras (Manguarian, 1994)

The Francisco Morazan hydroelectric project, completed in 1985, provides 300 MW of electricity to the nation of Honduras and neighboring Nicaragua. The project entails a 226-m high double curvature arch dam across the narrow box canyon that gives the dam its popular name, "El Cajon."

Problems became evident at the $775 million project as soon as the reservoir was filled in 1986. The weight of the water caused cracks to form in the cement grout curtain behind the dam resulting in seepage through, and erosion of, pockets of clay in the limestone under the dam. By 1993, seepage exceeded 1,600 liters per second and was discolored with clay, indicating that the fissures and karsts were getting larger.

Grout injection was decided upon, and special drilling and injection machinery were designed to withstand the pressure produced by the reservoir behind the dam. Hundreds of holes, some up to 250-m deep, were drilled from the dam base into the limestone and many grout mixes tried. However, most of the grout ended up in the powerhouse sump.

Geologists and engineers decided that larger objects were needed to plug the fissures and karsts, but there was no well-sorted, round gravel of this size in the vicinity. Instead, wooden and plastic toy balls filled with concrete were used. Altogether, the engineers injected 8,650 plastic and wooden balls into the hundreds of holes they had drilled. But while the balls remained in place, they didn't hold back enough of the cement grout. To combat this, rolled up polypropylene feed sacks were injected into the holes – over 25,000 of them. By April of 1995, leakage had been reduced to less than 100 liters per second and hydrostatic pressure beneath the dam was cut by over 60 percent, allowing the reservoir to refill and the powerhouse to generate fully.

No. 15 Insufficient Spillway Capacity and Seismic Instability, Buttress Dam Big Dalton Dam (MWH, 1999)

Big Dalton Dam, built in 1929, is located in Big Dalton Canyon about four miles northeast of Glendora, California. The project provides flood control and water conservation. The 146-ft high dam has six arch barrels and five hollow buttresses with a gravity section on each abutment. The arch barrels and buttresses are constructed of reinforced concrete with a crest length of 480-ft.

In 1975, the dam was evaluated for safety and structural integrity under earthquake, water, silt and dead loadings. A project report was prepared that presented preliminary recommendations for remedial work. The dam was again analyzed in 1994-1995 using state-of-the-art techniques, including a finite element analysis, and additional field geotechnical data was collected to improve the site characterization. The analyses concluded that the dam did not meet specified stability criteria during predicted seismic events and the project did not possess adequate spill capacity.

Designs were prepared to address the seismic stability issues and spillway capacity. The seismic stability was increased by infilling the downstream area between buttresses with a concrete gravity section (Figure 4.3-13). This section fill extended to a point just upstream of the downstream end of the buttresses, preserving the general architectural appearance of the Eastwood Dam design. Originally designed as an RCC placement, the contractor offered a value engineering proposal to reduce project cost if conventional concrete could be used based on local availability and cost of materials. This proposal was accepted by the designer, the owner, and the state dam safety office.

Spillway capacity was increased by raising the parapet wall on all but the two center arches. In a flood, these two arches would function as an uncontrolled semicircular overflow weir (Figure 4.3-12). The top of the new gravity section beneath the two overflow arches was shaped with a flip bucket to throw the overtopping flows clear of the downstream toe.

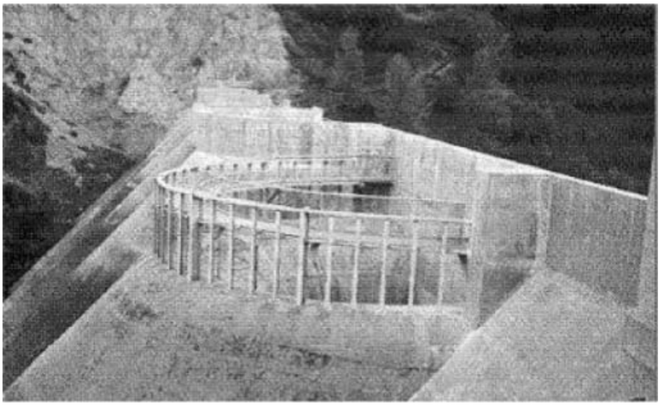


Figure 4.3-10 Big Dalton Dam (courtesy of MWH)

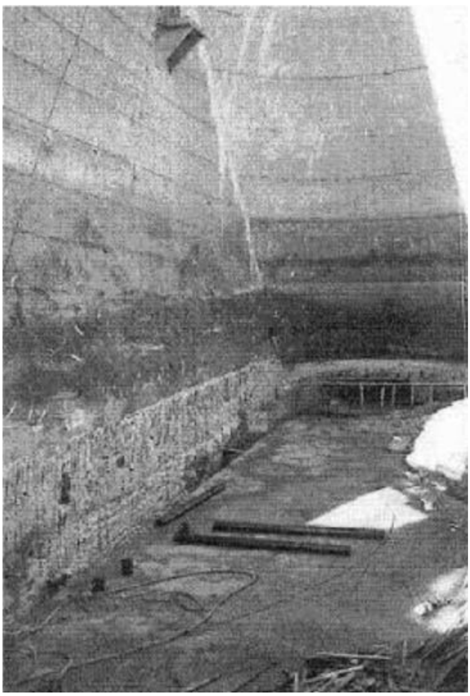


Figure 4.3-11 Big Dalton Dam (courtesy of MWH)

Figure 4.3-12 Boundary Dam (courtesy of MWH)

No. 16 Flooding of Drainage Gallery and Insufficient Stability Leesville Dam (Kleinschmidt Associates, 1998)

The Leesville Hydroelectric Project is located in Leesville, VA. The project consists of a 980 foot long concrete gravity dam with integral powerhouse. The dam has a structural height of 91 feet, to develop the site's 72 feet of gross head for two 20 MW turbines. The dam forms the lower pool for the 688 MW Smith Mountain Pump Storage Project. The dam is founded on greenstone, an igneous rock, and has a foundation drain system with drainage gallery that is used to reduce the uplift pressure beneath the structures.

The project's construction drawings indicated that the foundation drains were designed to reduce the uplift pressure from full headpond pressure at the heel, varying linearly to tailwater pressure at the foundation drains which are located at 9.5 feet downstream from the dam's heel. From the foundation drains, the uplift pressure was taken as tailwater pressure. The construction drawing also indicated that the uplift pressure was applied over 50% of the base of the structure; the dam was designed for a headpond level that was equal to the maximum normal pond (three feet below the crest of the dam); and for determining the dam's safety of factor, a foundation cohesion of 450 psi was used along the concrete – rock interface.

As part of the FERC's dam safety inspection program, and based on current design criteria, a number of concerns were expressed with the dam's stability and factors of safety: 1) foundation monitoring data indicated that the efficiency at the drains under normal pond conditions was 90% of difference between headpond and tailwater pressures (not 100% as designed); 2) current design standards require applying the uplift pressure beneath the full area of the dam's base (not 50% of the area as designed); 3) the use of a high value of cohesion (450 psi used in the design) cannot be assumed without substantiation, and historic field data did not indicate high cohesion values were possible; 4) current guidelines for the high hazard dam required that the dam be stable for floods up to the Probable Maximum Flood (PMF) rather than the maximum normal pond; 5) the PMF at the dam would overtop the non-overflow structures by 15 feet.

For this project the FERC would allow a remediation of the dam's stability with the following assumptions: 1) no cohesion used in calculating the dam's factor of safety; 2) friction factor of 1.0; 3) minimum factor of safety of 1.5 for the critical loading condition; and 4) the use of a 90% drainage efficiency (measured at the vertical drains) for normal pond and flood loading conditions was acceptable, as long as the remedial design analyses indicated that there would be no development of a tension zone (cracked base) beneath the structure.

Stability analyses indicated that the critical loading condition was the PMF, and the analyses also indicated that if a 3.5-foot high flood parapet wall on the non-overflow sections of the dam were removed, it would lower the PMF headpond further and reduce the hydrostatic pressures against the dam. A site specific PMF study was performed to upgrade the flood discharge requirements at the dam, and it determined that at the PMF headpond would be 8 feet higher than normal pond (rather than 15 feet as previously calculated), with the removal of the parapet wall, alone lowering PMF headpond level by over one-foot. The revised PMF indicated that the headpond and tailwater levels would be 8 feet and 42 feet higher than the levels used in the design of the dam; the headpond would overtop the non-overflow portion of the dam by 6 feet; and the tailwater level would be 18 feet above the invert of the adit into the dam's drainage gallery.

To increase the stability of the dam, the parapet wall was removed and forty-seven tendon type rock anchors were installed with a design load that varied from 493,000 lbs to 1,514,000 lbs (average load was 1,135,000 lbs). The anchors from 84 to 126 feet in length, and were fully bonded and encased in a seamless corrugated sheath that extended from the tip of the anchor to the anchor head. The use of fully bonded anchors eliminated the need for long term monitoring of the anchors for movement or relaxation of loads.

To prevent the flooding of the drainage gallery and maintain the operational effectiveness of the foundation drains under flood conditions, watertight bulkheads were installed over three vertical access shafts into the gallery and at the gallery adit, and all conduits and other penetrations into the gallery were sealed to reduce inflow leakage, due to the elevated tailwater level. A back up power generator for the drainage galleries dewatering pumps was installed at an elevation above the level of the PMF headpond, and provided with a fuel supply that would be sufficient for three days of continuous operation under full load. The power leads to the dewatering pumps were rerouted into the dam and sealed to be watertight. All pump controls were placed in watertight enclosures or replaced with submersible equipment. The three 450-gpm pumps had been replaced in the recent past, and their discharge capacity and discharge head were judged to be sufficient for continuous operation under PMF conditions. Discharge valves and piping were upgraded to prevent backflow into the gallery in the event of a pipe failure.

The Project's operating plan was modified to identify when the watertight bulkheads were to be closed and locked, and what other procedures needed to be performed to insure that the drainage gallery remained watertight and the pumps operational.

The stability remediation and upgrade of the drainage system did not provide any opportunity for the improvement of the operation of the hydroelectric project, although the site specific PMF study and "minor" modifications such as the removal of the parapet wall resulted in reducing the costs of the remediation by 67%.

b) Embankment Dams

No. 1 Foundation Liquefaction
Lopez Dam (MWH, 2002)

Lopez Dam is located on Arroyo Grande Creek, about 12 miles southeast of San Luis Obispo, California. The San Luis Obispo County Flood Control and Water Conservation District owns and operates the dam and reservoir. The 166-foot high embankment dam, completed in 1968, impounds a 52,500 acre-foot reservoir at the spillway crest elevation of 520 feet. As shown in Figure 4.3-13 below, it was built as a zoned earthfill dam with a central clay core, a gravel upstream shell, a random fill downstream shell, and filter zones between the core and the shells.

Figure 4.3-13 Lopez Dam
(courtesy of MWH)

Both the upstream and downstream shells have slopes of 3 horizontal to 1 vertical (3H:1V). The slopes include a 100-foot wide berm at elevation 450 feet on the upstream shell and a 140-foot wide berm at the same elevation on the downstream shell. The core extends to foundation rock, but the shells of the dam were founded on river alluvium, which has a maximum thickness of approximately 110 feet.

Engineering studies of the safety of the dam performed in the early 1990's found that the dam would be unsafe if shaken by the Maximum Credible Earthquake (MCE), a Magnitude 7 event on the West Huasna fault located less than 1 mile from the dam. The MCE would produce much stronger ground shaking than was considered during the original design in the 1960's. The engineering studies also indicate that the alluvium beneath the shells of the dam could lose strength due to liquefaction during earthquake ground shaking. The potential for liquefaction of the foundation alluvium could lead to breaching of the dam and uncontrolled release of the reservoir, and significant flooding of the areas and communities downstream of the dam.

The California Division of Safety of Dams (DSOD) has the jurisdiction for review of dam safety for non-federal dams in California. The DSOD reviewed the results of safety analyses of the dam performed between 1992 and 1996. Based on their review, they directed the County to remediate and upgrade Lopez Dam to current standards of safety. In response to this directive, the County prepared a plan for the design and construction of remedial measures.

Seismic remediation alternatives for Lopez Dam were developed and screened in a two-step process. The first step consisted of identifying potential remediation approaches. The second step included refining the identified alternatives and selection of a preferred (recommended) alternative. The remedial approaches considered were:

- Construction of a buttress on one or both sides of the dam, resting on existing foundation alluvium;
- Foundation improvement, combined with construction of a buttress on a strengthened (improved or removed) foundation;
- Remediation of the upstream shell of the dam, without reservoir draining or lowering;
- Construction of a new dam located immediately downstream from the existing dam, designed and constructed to current standards of practice; and
- A "no-fix" alternative, consisting of the minimum required construction and measures to obtain regulatory acceptance of the dam (i.e., permanent reservoir lowering).

The foundation "improvement" measures considered included (1) complete excavation to bedrock and replacement with compacted fill and (2) in-situ ground improvement of existing alluvium.

The recommended approach for seismic remediation of Lopez Dam included a downstream buttress and in-situ ground improvement of the foundation.

The buttress included a widened dam crest to provide a sufficient width of dam to retain the reservoir even in the event of large upstream slope deformations and to limit downstream slope deformation of the dam resulting from the design earthquake. This general approach was selected because it meets key criteria including no reservoir draining.

The stone column alternative was selected for in-situ foundation improvement and protection against liquefaction because it had the least cost, is environmentally responsible, and had the shortest construction period. It also did not require lowering the reservoir below the maximum restriction level of elevation 510 feet. Other foundation improvement methods considered for this project, but rejected, included deep dynamic compaction (alluvial soils too thick), vibrocompaction (alluvial soils too fine-grained), and deep soil mixing (verification issues).

The selected alternative consisted of three steps: removing a portion of the downstream shell and excavating a portion of the foundation alluvium; installing stone columns to bedrock to strengthen the foundation alluvium; and reconstructing the downstream shell of the dam with a wider dam crest and a buttress.

To accommodate the variable foundation materials, the construction specifications provided for two separate column construction criteria. In soils where densification occurred readily (typically granular materials with low fines contents), column construction progress was governed by the maximum operating amperage level of the vibratory probe. In fine-grained soils (typically sandy silts), the probe did not build up peak amperage levels. Therefore, in these types of soils, the column construction progress was controlled by a maximum 72-inch column diameter requirement.

The specifications also allowed the engineer to make revisions to the final stone column spacing and sequencing for production, based on the test section results. The test sections and subsequent spacing and sequence revisions are covered in a subsequent section.

The construction of a stone column is completed in two stages, Penetration and Fill as shown in the Figure below. In the Penetration stage the vibratory probe is advanced to the required depth by a combination of the probe's vibratory action, the weight of the probe, and if necessary, the use of air jets attached to the side of the stone feed pipe. In some instances it was necessary to pre-drill certain soil types to aid with penetration.

For the Fill stage, a known weight of stone (a charge) is fed into the pressure vessel. As the vibratory probe is raised slightly (stroke height) from the lowest penetration depth, stone is forced from the pressure vessel through the feed pipe, and discharged at the tip of the vibrator. The vibrator is lowered through the previously discharged stone, thus displacing the stone laterally. This procedure of raising the probe, discharging stone, and repenetrating the stone is continued until all the stone in the pressure vessel is used. The portion of a column constructed with one charge of stone is termed a "lift". A column is completed by re-charging the pressure vessel, and repeating the cycle.

Construction staging was particularly important at this site, as the stone column work areas were only accessible after sequential stages of foundation excavation were completed. A total of 177,000 lineal feet of columns were installed in the dam foundation. The depths of these columns ranged from 15 feet to 95 feet, although the specifications required the column equipment to be capable of 100-foot depths.

An extensive program of penetration testing was conducted at the site to verify the results of the stone column construction. The testing included both Standard Penetration Tests (SPTs) and Becker Penetration Tests (BPTs). Prior to stone column construction, but after foundation excavation was completed, a series of about 30 BPT borings were drilled over the entire treatment area, through the alluvium and into the top of bedrock. Companion SPT borings were conducted adjacent to about half of the BPT borings, to obtain soil samples for laboratory testing (primarily gradation) and to provide a data set for correlating the SPT and BPT results.

Seismic stability and deformation analyses were also performed following completion of each main group of stone column blocks to validate that the as-constructed stone columns would provide the required ground improvement. The results of the post-construction deformation analyses at Lopez Dam verified that the as-constructed stone columns provide sufficient strengthening of the foundation soils. The foundation improvement combined with the stabilizing buttress achieved the project goal of limiting dam crest deformations under seismic loading to acceptable values.

Constructing the seismic strengthening measures at Lopez Dam to prevent liquefaction failure was a complex undertaking, presenting a number of significant challenges and risks. Variable and difficult foundation conditions necessitated revisions to the stone column spacing and sequencing. Recognition and understanding by the owner, engineer, and contractor of the risks inherent in complex subsurface construction projects was critical to the success of the project. Clearly, effective teamwork by all parties was essential to achieving a successful outcome.

With a total project cost of about \$30 million (including construction, engineering, environmental studies, financing, management, etc), the selected design approach for seismic strengthening of Lopez Dam saved the owner approximately \$10 million in comparison to a conventional buttress founded on bedrock. This design also allowed the project to be built while the reservoir remained in service. Completed in 2003, the Lopez Dam project represents the first DSOD approval of stone columns for remediation of a large earth dam in California. The project is now back in unrestricted operation, once again able to provide full water supply and flood control benefits for the local communities.

No. 2 Toe Erosion at Outlet Works
Milford Dam (Dridge & Matthews, 1997)

Milford Dam is a US Army Corps of Engineers Dam located on the Republican River in east central Kansas. The dam consists of a zoned earth and rockfill embankment approximately 6,300 feet long and 110 feet high. The foundation is comprised of a thin layer of alluvial silt overlying a thick sand deposit. Underseepage is controlled by an upstream blanket and 72 relief wells along the downstream toe. The outlet works includes a tower, gated single 21-foot high horseshoe conduit, and stilling basin. The outlet channel extends from the stilling basin to the natural river channel, a distance of slightly more than 1 mile. The trapezoidal channel has an average bottom width of 200 feet and riprap protection on the side slopes. The spillway is a limited service, uncontrolled, chute type excavated in the right abutment. Milford Dam was constructed and is operated by the Kansas City District, U. S. Army Corps of Engineers.

The project experienced record pool levels in July 1993 that required large flood releases through the outlet works and several days of uncontrolled flow though the spillway. Releases through the outlet works caused significant erosion of the channel banks downstream of the stilling basin.

The left bank soils consist of an upper layer of clayey silt approximately 3 feet thick, overlaying layers of fine to medium silty sands. After loss of the riprap slope protection, the soils eroded at an extremely high rate. Within 10 hours, a large scour hole approximately 600 feet in length and extending a maximum of 50 feet into the bank, had developed, along the left side. The primary concern was that the erosion could lead to piping of the dam's foundation as the sand exposed in the eroded channel bank is continuous beneath the embankment. Underseepage through this layer is generally controlled by the relief wells along the downstream toe. The erosion provided an unprotected (unfiltered) exit point for seepage that was much lower in elevation than the relief well outlets. Continued erosion would have worsened the situation by exposing more foundation material and shortening the seepage path. Visual surveillance for evidence of piping was difficult because most of the bank was underwater. The right bank experienced similar erosion, but it occurred several hundred feet downstream and did not represent a piping threat.

Emergency repairs involved regrading the eroded bank and placing large stone (48 inch diameter) over filter fabric in the most critical area. Rock was end dumped over the eroded bank for the remainder of the scour hole. Over a period of 5 days, approximately 4,500 tons of rock was delivered to the site. Two thirds of this material was used for the repair, while the other one third was stockpiled for future contingency. The repair allowed flow releases to be increased to evacuate the flood pool and provided the needed interim slope protection until a permanent repair could be initiated.

After the reservoir level was reduced, the outlet channel was dewatered to allow a detailed inspection. Immediately downstream of the stilling basin the channel bottom was a limestone layer that crossed the channel at a significant skew and was lined with derrick stone. The derrick stone had been placed in the early 1970's to stabilize a deep scour hole. The remainder of the outlet channel, as well as the river downstream, has a sand bottom.

The sand bottom portion of the channel has been degrading since the dam was placed into operation. However, the rock immediately downstream of the stilling basin remained intact. This resulted in a very high gradient downstream of the rock bottom. In fact, the rock bottom was acting as a broad crested weir and controlled discharges emerging from the stilling basin. As the discharges passed over the weir, supercritical flow was approached.

The left bank had been originally constructed to a 3H:1V slope protected with a 36 inch layer of riprap overlying a 12 inch thick bedding layer. The riprap had been specified with a maximum size of 1400 lbs, between 30 and 60 percent lighter than 700 lbs, and no more than 10 percent smaller than 2 inches. According to current criteria, this rock is significantly undersized for the observed velocities. It is likely that there was an isolated thin spot or pocket of smaller rock that was removed by the high velocity flow, allowing the rest of the riprap to unravel. Another factor that may have contributed to the riprap failure, but is not believed to be a primary cause, is that the bedding material did not meet current filter criteria.

Several alternatives were evaluated to restore the channel to allow safe release of the design discharge, maintain adequate tailwater at the stilling basin with anticipated future river degradation, and provide adequate filter protection against underseepage at the exit point along the left bank. The most cost effective alternative was to deepen a portion of the channel by excavating the rock bottom immediately downstream from the stilling basin. This would effectively reduce channel velocities to an acceptable level and enable riprap to be used on the side slopes. The deepening, however, was complicated by the right bank, which was excavated into a high rock hillside.

Significant savings could be realized by shifting the deepened channel to the left, avoiding a large amount of additional rock excavation. This would result in an unsymmetrical outlet channel immediately downstream of the stilling basin. In order to evaluate the hydraulics of such an unsymmetrical channel and to investigate required stilling basin tailwater depth, a physical model study was constructed, which ultimately showed that the unsymmetrical, deepened channel would work. However, the stilling basin performance was found to be very sensitive to minor variations in tailwater elevation. The stilling basin performance was fine for the post-constructed configuration, but when tailwater was lowered just a few feet to simulate future downstream channel degradation, the basin no longer adequately dissipated the energy. In order to limit channel degradation and stabilize future tailwater, a channel stabilizer structure was required.

The selected repair consisted of deepening the outlet channel immediately downstream of the stilling basin, restoring the riprap on both channel banks including a significant rock toe with multiple bedding layers to filter seepage, and the construction of a channel stabilizer downstream of the stilling basin. The channel stabilizer is a cellular sheet pile structure extending across the channel. The sheet piling was driven to rock with the finished top nearly level with the existing channel bottom elevation. Large rock was placed both upstream and downstream of the sheet pile structure to form a hard point in the channel invert. This option was selected over an expensive drop structure, with consideration given to the trade-off between initial costs and long-term cost of maintaining the channel.

Construction of the channel repair proceeded on budget and on time. A test release was made and the results of this test indicated an acceptable performance of the channel repair.

No. 3 Insufficient Spillway Capacity and Seepage
He Dog Dam (MWH, 1995)

He Dog Dam is an earthfill embankment with a height of 46 feet, and a 1300-feet long by 14-feet wide crest. The original service spillway has a 50-feet-wide chute spillway and a remote grass-lined emergency spillway on the right abutment. As part of earlier dam safety investigations, the spillway was determined to be undersized, excessive seepage was identified at the left end of the embankment, artesian pressures were noted in the dam's foundation, and the low-level outlet works were found to be inoperable for emergency use.

In 1991, conceptual and final designs for the rehabilitation of the dam to address all dam safety concerns were prepared. Final design for rehabilitation of the dam included; a new 250-feet wide roller-compacted concrete service spillway; raising the crest of the embankment 11 feet; a partial chimney drain seepage collection system; and wick drains for relief of the artesian pressures; and the installation of a new pipeline and downstream control structure for operation of the low-level outlet works. Construction was completed in 1995.

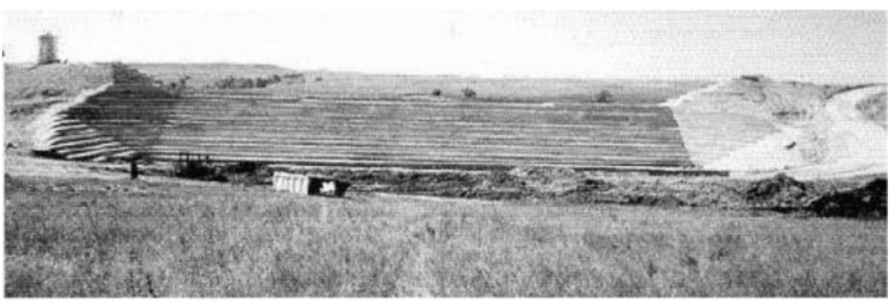


Figure 4.3-13 He Dog Dam, New Spillway (courtesy of MWH)

No. 4 Insufficient Spillway Capacity
Middle Branch Dam (MWH, 2000)

The City of New York constructed Middle Branch Dam in 1878 for water supply. Middle Branch Dam is a homogenous earth embankment with a masonry corewall, has a height of 95 feet and a crest length of 510 feet. The existing spillway is a 100-feet wide rock-cut channel and has a capacity of 10,600 cfs. A valved, low-level outlet works is located in the left abutment and includes a stone masonry intake structure, pressurized and non-pressurized brick-lined tunnels, two 30-inch pipelines and a valve chamber.

A study was performed for the City of New York to determine if the dam met modern dam safety requirements. The study included hydrologic and hydraulic analysis, geotechnical investigations and analysis, and visual and underwater inspections. Results of the study showed that the existing spillway capacity was approximately one third of the inflow design flood. The low-level outlet works was also in need of rehabilitation.

A rehabilitation plan was developed that included; a new auxiliary spillway; rehabilitation of the intake tower; replacement of the valve chamber, piping and all required appurtenances; and site improvements comprising a floodwall in the left abutment, reconstruction of two access roadways and a new boat ramp.

Design of the new auxiliary spillway consisted of a two-bay fuse plug on the crest of the embankment, roller compacted concrete (RCC) placed on the crest and downstream slope to prevent erosion, and innovative starter notches for the fuse-plug constructed of half-round corrugated metal pipe. The RCC and fuse-plug are covered with topsoil and seeding to maintain the appearance of the dam as part of historic preservation requirements. An upstream sheetpile cofferdam and a downstream earthfill cofferdam were also designed.

No. 5 Foundation Liquefaction
Wickiup Dam (Bliss, 2003)

Wickiup Dam was constructed between 1939 and 1949 on the Deschutes River near Bend, Oregon. It is a 100-ft high rolled earthfill embankment with a riprapped upstream face and rockfill downstream face. Analyses indicated that low-density materials are present in the foundation of the left abutment dike that is susceptible to liquefaction under strong ground shaking. The dike foundation consists of horizontal layers of fluvial sands and gravels, volcanic ash, and lacustine silts and clays.

A corrective action scoping study of alternatives, completed in January 1999, indicated that jet grouting of these foundation materials could potentially be the least cost alternative to stabilize the soils and prevent failure of the dam under strong ground shaking. However, little historical information was available from which to judge the potential cost and effectiveness of the jet-grouting plan, so a test section was constructed.

The test section was completed in late January 2000 and was followed by an extensive field investigation program, which ultimately showed that the jet grouting test section was successful. As such, final designs for the jet-grouting alternative were prepared and construction began in July of 2001. In addition to the jet grouting, a blanket filter and drain system along the foundation contact of the berm were designed and implemented.

No. 6 High Underseepage Gradients
Miami Conservancy District (MWH, 2005)

The Miami Conservancy District owns and operates five dry flood control dams (Lockington, Taylorsville, Englewood, Huffman, and Germantown Dams) in the Great Miami River basin near Dayton, OH. All of the dams are hydraulic fill structures, 1,200 to 6,400-feet long, constructed between 1918 and 1922. None of the dams impound permanent reservoirs though the dams have stored water over 1,500 times during their 80-year history with maximum reservoir depths of 54 to 69% of the design reservoir depth.

The dams are founded on glacial outwash deposits ranging from 40 to 240-feet deep. During storage events, significant flows from relief wells installed near the downstream toe of the dam and seepage from low areas downstream were recorded. Instrumentation indicated significant rises in observation well readings with rapid response to reservoir levels indicating that high underseepage gradients might occur near the toe of the dam producing unstable conditions.

Finite element underseepage studies were conducted to estimate gradients at various reservoir levels and to establish the maximum reservoir levels which the dams could safely impound. Cross sections for the analyses were selected and idealized based on the study of foundation conditions, recorded seepage and relief well flows, and profiles of the entire range of foundation conditions. Ranges of coefficients of permeability for the foundation material types were determined on the basis of data from drill hole permeability testing and well pump-out tests.

The analyses indicated that extremely high gradients exist at high reservoir levels, which have the potential to move fine foundation materials into coarser materials (piping). Also, the analyses indicated that the capacity of the relief wells could be exceeded under flood conditions.

In response to the conclusions of the analyses, underseepage remediation systems were designed and installed. The systems included additional relief wells, a downstream drainage collection trench, and a passive weighting berm complete with an underseepage collection and discharge system.

In addition to these systems, the corewalls of each dam were to be extended vertically within the cross section of the dams. Slurry wall technology was employed to perform this work with slag cement bentonite implemented at one of the dams to provide a higher strength core wall due to the presence of a State Highway located atop the dam.

No. 7 Seismic Instability
Diversion Dam (Findlay & Rabasca, 2003)

Diversion Dam, located in Croghan New York is an embankment dam that is a component of the Soft Maple Hydroelectric development. The development is part of the Beaver River Project, owned and operated by Brookfield Power New York. The dam is under the regulatory jurisdiction of the Federal Energy Regulatory Commission (FERC). The dam is approximately 900 feet long at the crest and is about 70 feet high, and has a high hazard classification under FERC Guidelines. The dam was constructed between 1924 and 1925 using hydraulic fill method. The hydraulic fill method of embankment construction, commonly used in the early half of the 20th century, generally results in looser density fill than would otherwise be achieved by modern compacted lift-fill placement methods. Because of the loose character of the fill, saturated portions of hydraulic fill dams may be susceptible to liquefaction triggered by seismic shaking.

Diversion Dam was first analyzed for seismic stability in 1981. At that time, the seismicity had been characterized as a 0.1g peak ground acceleration (PGA), resulting from an earthquake of magnitude (M) 4.75. By the early 1990's, the methods of liquefaction analysis had been adapted to sloping ground conditions. Also, the seismicity of the site had been redefined using an updated understanding of attenuation of eastern earthquakes and probabilistic methods. This resulted in the maximum credible seismicity of the site being redefined as 0.17g PGA resulting from an earthquake of M 5.3 to 5.8. Because of these changes, and as requested by the FERC, the potential for liquefaction at Diversion Dam was re-evaluated.

As a result of review of the subsurface exploration borings made for previous seismic assessments, it was determined that the drilling and sampling methods used could have resulted in disturbed test zones. This cast some doubt as to the accuracy of the resulting standard penetration test (SPT) N values, which were the basis of evaluating the seismic stability of the dam. As a result of this concern and the review of 1920s vintage construction photos that showed a cofferdam on the upstream side, additional borings and SPT testing were warranted to better define subsurface conditions.

A program for further investigation of the dam consisting of thirteen borings on the upstream and downstream sides of the dam was conducted including borings drilled over the water from a barge. The procedures for conducting the SPT tests were those outlined by Seed, *et al*, (Seed 1984). Both an automatic hammer and safety hammer were used in the investigation. The hammer energy for both systems was measured at the start of the field program. The new STP tests resulted in N-values greater than those obtained from previous exploration programs. The project team attributed this to the drilling procedures used, which minimized disturbance during the drilling process. The SPT sampling procedures for this project and comparisons with previous subsurface data is the subject of a paper by Rabasca and Findlay (1999).

Using the data available from the new borings as well as the information from previous studies, a detailed seismic stability analysis of Diversion Dam was conducted. Because of differing conditions along the longitudinal axis of the dam, the liquefaction analysis was done for three dam cross sections as the first part of the seismic analysis. The liquefaction analysis involved a series of inter-related steps and included a finite element seepage analysis, a finite element static stress analysis, a finite element dynamic response analysis, and a triggering analysis. The finite element analyses result in data that is necessary for conducting the triggering analysis. The triggering analysis determines if and where in the dam cross section liquefaction would occur under a specific seismic event. The Diversion Dam liquefaction analysis considered three earthquake records and three cross sections, for a total of nine separate liquefaction analyses. In addition, numerous sensitivity analyses were also performed to assess the impacts of a range in N-values for various zones. Papers by Findlay and Rabasca (2003) and Rabasca and Findlay (2003) discuss these analyses in more detail.

The triggering analyses showed that under the assumed seismicity analysis, the puddle core would undergo liquefaction or significant pore pressure development. The upstream shell was predicted to develop excess pore pressure over many elements, and shallow portions of the upstream shell would liquefy. On the downstream slope, the lower (deeper) downstream shell soils were predicted to undergo liquefaction in the outer elements, and excess pore pressure was found to develop between the toe and the puddle core. The results of the triggering analysis were used to develop a model for post-earthquake slope stability analyses. This analysis is a conventional slope stability analysis which incorporates reduced material strengths, as appropriate, for zones of the dam that are shown by the triggering analysis to liquefy or develop seismically induced pore pressures. The downstream factor of safety at the maximum section did not satisfy the required minimum factor of safety of 1.3; therefore remediation on the downstream slope was necessary, according to the analysis guidelines stipulated by the FERC. As a final step in the seismic assessment, a deformation analysis, involving the direct integration method presented by Newmark (1965) was conducted. Estimated deformation was found to be significantly less than 2 feet in all analyzed cases.

Evaluation of the results of the seismic stability analysis involved coordination with the FERC and resulted in the determination that the stability of the downstream slope of the dam should be improved with regard to seismic stability. Remediation alternatives considered included stone columns, deep soil mixing, vibroflotation, and a buttressing stability berm. The most cost effective remediation of the downstream slope to improve the seismic stability factor of safety was found to be construction of a berm at the toe of the main portion of the dam. Analysis of supplementary sections of the dam at the left abutment indicated that a berm was not necessary in that area; nonetheless, a smaller berm was added as part of an overall seepage remediation, subsequently described.

As part of the remediation of the dam, drainage systems were incorporated to improve collection and discharge of seepage at various locations in the downstream slope and abutments. A history of seepage problems existed on the left abutment of the dam, dating back to at least 1980, and likely before then. The seepage at the left abutment is a result of the fact that the concrete core wall was not constructed on this end of the dam, as well as the fact that the bedrock surface was higher. Seepage related issues were also present to a lesser degree at the right abutment groin, and along linear horizontal areas in the center area of the main section of the dam. These linear seepage areas were at two distinct elevations, higher on the dam slope. The linear seepage outbreaks were believed to be related to the presence of isolated thin silt layers, interbedded within hydraulic fill lifts. The seepage was believed to be caused by seasonal rains infiltrating down through the shell soils until the silt layers were encountered.

Construction of the seismic and seepage remedial measures commenced in August of 2002, and were completed by November 2002. Figure 4.3-18 shows a section through the dam cross-section, and shows the relative size of the stability berm that was necessary to adequately increase the post-earthquake factor of safety to levels acceptable to the FERC. Fill material for this berm was from a local pit, and was transported to the site and compacted in lifts. Approximately 15,000 cubic yards of material were required to construct the seismic stability berm.

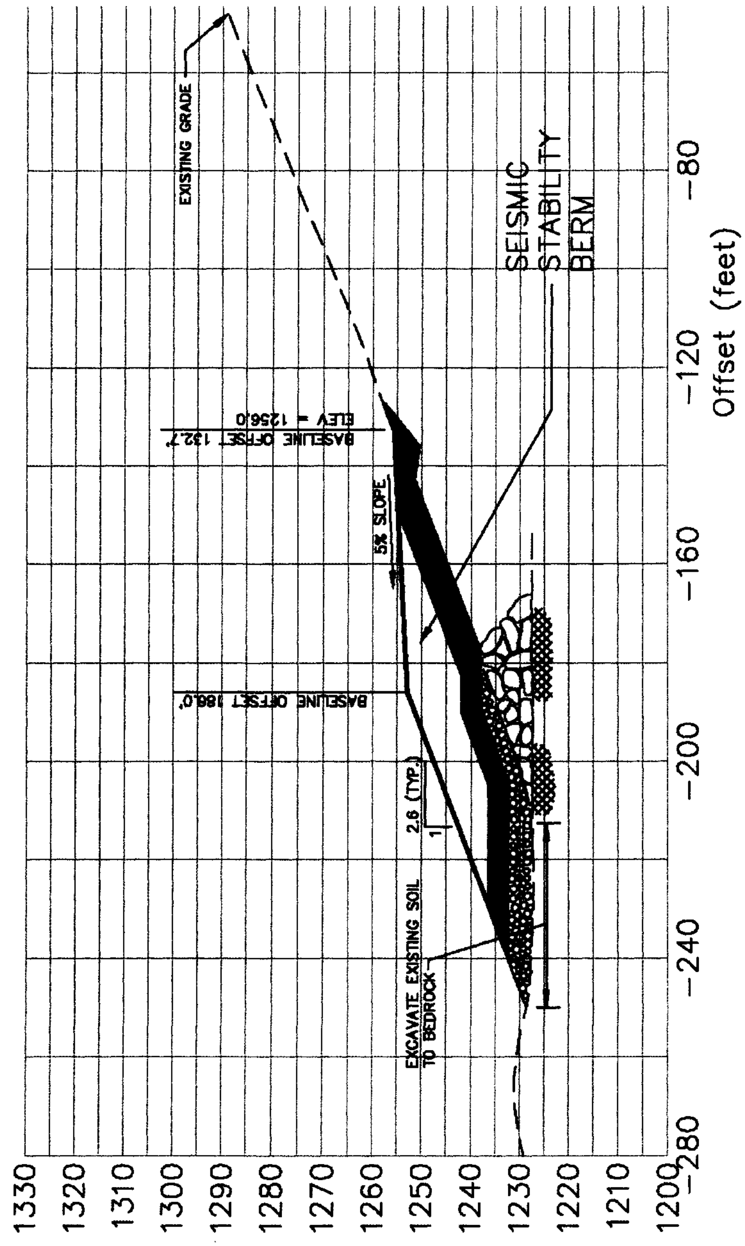


Figure 4.3-15 Diversion Dam Seismic Remediation (courtesy of Findlay & Rabasca)

The lowest lift of the berm consisted of 6-inch ballast-type stone to allow free drainage from the stone berm at the toe of the existing dam. Reverse filters were placed above this stone layer to mitigate infiltration of fines from the overlying common borrow berm materials.

c) Timber and Masonry Dams

No. 1 Timber Crib Dam Deterioration
Wallowa Falls Project (Greenman *et al*, 1997)

The Wallowa Falls dam is a timber crib constructed in 1921 on the East Fork of the Wallowa River near the town of Joseph, Oregon. The 23-foot high dam is part of the Wallowa Falls Hydroelectric Project, which generates 1100-kW from the less than one acre-foot of storage through the 1173-feet of available head. The dam was constructed with a sloping upstream face, a vertical downstream face, and had been covered with plywood and plastic to reduce leakage. Maintenance of the face, together with significant deterioration of the timber cribs, made it necessary to undertake major structural revisions.

Because of the dam's wilderness location, it was desired that the repair be accomplished with minimum disruption to the essentially pristine environment. Several alternatives were considered including replacement of the dam with an RCC structure. This however proved not feasible due to the dam's location, lack of adequate access and materials, and economics. Another alternative involved the placement of an asphaltic concrete overlay on the sloping upstream face but was ultimately rejected, as the overlay would be weakened by the continuing deterioration of the underlying timber structure.

An entirely new rockfill dam was also considered. However, the cost of transporting large volumes of impervious core or intermediate transition zone filter material to the site made this alternative economically unattractive. Ultimately, it was decided to create a rockfill dam with a central impervious core that would utilize the upstream profile created by the timber cribs. The core was located immediately downstream of the vertical face of the existing dam utilizing the timber cribs as a cofferdam for construction. Also, with the core downstream of the crib structure, the logs are no longer subject to wetting and drying cycles, as they will be permanently wet thereby extending their useful life.

Several materials were investigated for use in the vertical core including reinforced concrete, RCC, hypalon, and an asphalt and rock matrix (Norwegian Method). A modified version of the method was selected which involved the use of a heavy polyester engineering fabric and plastic liner to retain the core matrix rock during asphalt placement. Since Oregon's fishery agency had used asphaltic concrete as liners for fish rearing ponds without deleterious effects on the fish, concerns regarding the vaporization of hydrocarbon derivatives and becoming suspended in the water were alleviated.

A bulk of the material needed for the construction, including the core rock, asphalt, landing mats, and reinforcing bars, had to be airlifted by helicopter to the dam site. The high elevation of the project limited the helicopter's lift capability but with a rate of 20 trips per hour, all lifts were accomplished in two days.

Repairs were completed in 1993 and annual inspections since then have revealed adequate performance, with displacements of less than 0.05-feet in any plane. The landing mat spillway has performed as anticipated, showing no sign of degradation.

No. 2 Masonry Dam Inadequate Stability & Spillway Capacity
Boyd's Corner Dam (Prendergast, 1991)

The Boyd's Corner Dam is located on the Croton River in upstate New York and was, for the first fourteen years after its construction, the highest masonry dam in the US. The 78-foot high dam is a cyclopean masonry structure with a granite-stone veneer built in 1870. In 1978, the 625-foot long dam was taken out of service due to safety concerns which affected about 15% of New York City's water supply.

Designated a historic structure by the New York State Historical Society and as such, the renovation of the dam had to preserve its historic value while upgrading the stability and increasing the spillway capacity to comply with new PMF estimates. The dam's original spillway was a rock cut through the left abutment, which offered about 10% of the required PMF peak discharge capacity.

Initially, a 200-ft long concrete overflow section in the center of the dam was contemplated but it was rejected by the Historical Society and the New York City Fine Arts Commission because the design would have significantly changed the dam's appearance, including the removal of up to 50% of the masonry veneer. Instead, a reinforced concrete, ogee-shaped, flip-bucket-type spillway was incorporated into the dam. Though just as wide as the first alternative, it was less visible and only 25% of the veneer had to be removed. The flip buckets direct the flow jet into a new plunge pool to dissipate the energy of the discharged water, preventing scour at the toe of the dam. A weir was also added to the existing spillway to further increase the spill capacity of the project.

To improve resistance to sliding, 21 post-tensioned anchors were added over the length of the dam. As the anchors imposed more stress than the masonry could take, a structural concrete cap was added to the top of the dam to uniformly load the dam body. The new concrete was incorporated into a bridge across the dam.

Finally, a grout curtain was installed within the dam to combat leakage. Though not common for masonry dams, a single row of holes were installed on the upstream face of the dam and the grout curtain was placed using carefully controlled pressures so that the masonry veneer was not damaged. The curtain was successful in reducing the dam's permeability by a factor of seven.

No. 3 Timber Crib Dam Deterioration
Centennial Mill Dam (Blanchette *et.al.*, 1997)

The Centennial Mill Dam is located on the Connecticut River between Dalton, New Hampshire and Gilman, Vermont. The first timber crib dam at this site was built in 1898. When the third timber crib dam washed out in 1927 a 4-unit powerhouse with integral intake and a significant paper mill complex remained. The dam, in combination with the powerhouse, provided process water, fire suppression water, and energy to the mill. The powerhouse is currently licensed for 4.85 MW.

In 1928, the fourth dam was built at the site and included a 111-foot long concrete gravity section founded on bedrock with a maximum height of 25-feet. A 213-foot long rock-filled timber crib section about 30 feet high was then built on top of glacial till, bedrock and the remains of the third dam. This method was likely adopted because of low cost, fast construction, the ability to build in the water, and to avoid excavating to rock which was quickly dipping down.

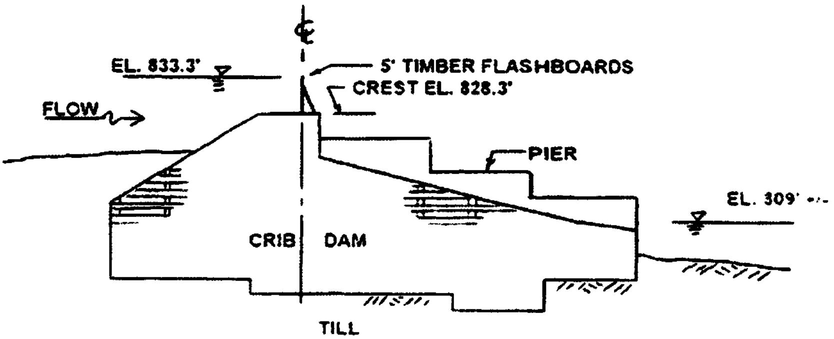


Figure 4.3-16 Centennial Mill Dam
(courtesy of Association of State Dam Safety Officials)

In 1979, a 45-ft section of crib dam next to the existing concrete section was removed and a concrete crest gate structure was constructed to reduce the frequency with which flashboards on the dam crest had to be replaced. During the early 1980's a new modern turbine/generator was installed.

Because of limited development along the riverbanks downstream, this dam is classified by the FERC as low hazard. In the early and mid-1990's, a diving team inspected the dam. The survey indicated that the existing concrete dam crest had eroded 0.3-feet with the downstream face showing erosion damage, and the timber crib dam crest had settled about 1 foot at the right end.

Conceptual designs were then prepared and costs estimated for options ranging from continuing the same maintenance program, to construction of a new concrete dam founded on till or on bedrock. Economic analysis of the options included; estimates of the time until the existing timber crib dam failed; costs to rebuild the dam after failure; expected maintenance costs of the options; and impacts to mill operations and energy generation. All options included installation of rubber dam crest controls on the entire overflow spillway to eliminate flashboard replacement costs and lost generation due to decreased head with flashboards not in place.

As the Owner wanted to ensure long-term viability of the structure and preserve the option to install a replacement/new turbine/generator unit in a future powerhouse, the construction of a new concrete dam and future intake founded on bedrock was chosen. In addition to the dam rebuild, the eroded existing concrete dam was to be refaced and unused sluiceways plugged with concrete.

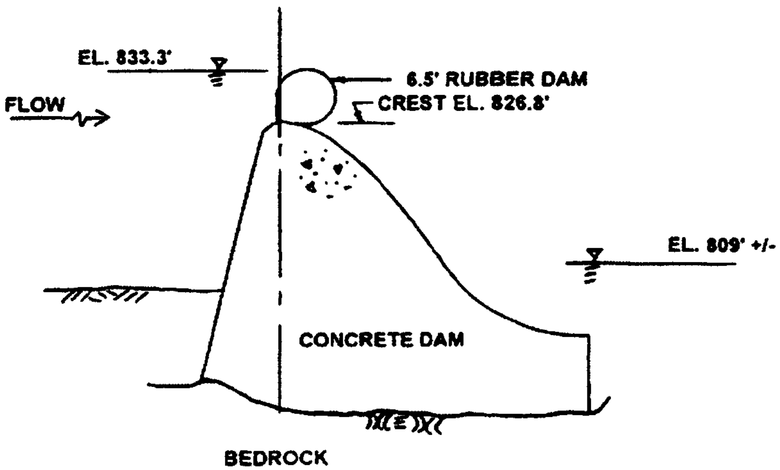


Figure 4.3-17 Centennial Mill Dam (courtesy of Association of State Dam Safety Officials)

Since the spillway crest length would be reduced 42-feet by the future intake, the new concrete dam crest was lowered 1.5-feet to provide the same spillway hydraulic capacity without raising the headwater. Provisions for a future downstream fish passage facility were also incorporated. Due to the size of the water passage provided for the future intake, stability analyses determined that post-tensioned rock anchors were required to satisfy the FERC factors of safety until the future powerhouse buttressed the intake.

The new works were completed, and the cofferdam removed sufficiently to allow generation to start again on only 105-days following initial shutdown. The permanent water retaining structures at the sites are now:

- Existing powerhouse foundation with voids filled with concrete, new upstream concrete cutoff wall and spread footing for future intake deck rehabilitation.

- New concrete bulkhead block and wall founded on bedrock with 30 inch diameter pipe for future downstream fish passage facility.
- Concrete future intake founded on bedrock with concrete plug, stone ballast and post tensioned anchors to be used as the upstream cofferdam, if a powerhouse is built in the future.
- New ogee shaped concrete dam (44 feet high maximum) founded on bedrock with new 6.5 feet high rubber dam crest controls.
- Leakage at existing crest gate structure discharge slab reduced.
- Existing concrete dam with new reinforced concrete facing, new 5 feet high rubber dam crest controls, and the sluiceways remaining from the 1928 construction plugged with concrete.

The owner, contractor, engineer, and permitting agencies consider the project's planning, design, and execution successful.

d) Reservoirs

No. 1 Flood Design Criteria Change
Upper Raquette River Project (Brookfield Power New York, 1989)

The Raquette River is located in Northern New York State and is a tributary to the St. Lawrence River. The river has 18 hydroelectric facilities (with their respective reservoirs) and a large storage reservoir at Carry Falls. In the prior PMF analysis conducted in 1984, the Raquette River basin was broken down into two sub-basins, the area from Raymondville to South Colton and from South Colton to Piercefield. This over-simplification only considered the storage effects of the Carry Falls Reservoir (1 of 19 reservoirs) within the analysis. Unit hydrographs from Piercefield to Carry Falls and from Carry Falls to Raymondville were developed. The local inflow above Carry Falls (Piercefield to Carry unitgraph) was routed through the Carry Falls Reservoir and combined with the local inflow of the downstream basin at Raymondville (Carry to Raymondville unitgraph). Peak flows at reservoir locations between Carry Falls and Raymondville were estimated by assuming the peak flows varied as a function of the intervening drainage areas.

The owner believed this approach overestimated the resulting PMFs since it ignored any attenuation of inflows and also implied that the resulting peak flow from the combining of the reservoir's outflows and the uncontrolled downstream local runoff are totally additive. The owner believed that the cumulative effect of the basin's reservoir storage and routing and combining of hydrographs would in fact significantly reduce the peak flows from major storm events. To test this theory, individual unit hydrographs for each reservoir's sub-basin were developed in the reanalyzed PMF.

The new PMF simulated the prior PMP using more advanced hydrological computer software (HMR52, HEC-RAS and HEC1) and a more thorough and accurate study of storm centerings. The analysis incorporated a more comprehensive depiction of sub basin delineation, headwater/tailwater-rating relationships at the reservoirs and incorporated reservoir storage and channel routing into the rainfall-runoff simulation model.

Drainage area delineations were selected to allow for the incremental runoff into each reservoir within the Raquette River basin. For each drainage area, unit graph parameters were estimated to allow for proper determination of local inflows. These local inflows were then routed through each reservoir and combined with the local inflows downstream. The Soil Conservation Service dimensionless unit graph method was selected since it is a common technique used whenever historical precipitation and flow data is unavailable, such that hydrograph reconstitution is not possible.

Also, in recent years, approaches in rainfall-runoff techniques have suggested sub-basin delineation on a much smaller scale (1 to 10 square miles). The SCS method is commonly selected in these techniques to simulate the basin response to rainfall.

This reanalysis resulted in reducing the PMF for the Upper Raquette River projects by about 30%. This more accurate assessment of PMF peak flows allowed the owner to meet Federal Energy Regulatory Commission (FERC) stability requirements without needless and costly dam stabilization modifications.

No. 2 Reservoir Liner Rehabilitation
Seneca Pumped Storage Station (MWH, 2000)

The 435-MW Seneca Pumped Storage Station is located on the Allegheny River in the Allegheny National Forest approximately nine miles east of Warren, Pennsylvania. The project is owned and operated by Cleveland Electric Illuminating, a FirstEnergy Company. The project uses the United State Army Corps of Engineers' Allegheny Reservoir, formed by the 179-foot-high Kinzua Dam, for the lower reservoir. The upper reservoir is located on a sandstone plateau about 800 feet above the river and is formed by a circular ring dike about one-half mile in diameter. The reservoir ring dike varies from about 30 to 120 feet in height with inside slopes of 2H:1V and variable outside slopes. Material for the dike was obtained from excavation within the reservoir and consists of clayey to silty sands, weathered sandstone and sandstone rockfill. The project is used to generate daily peak power, resulting in daily water level fluctuations of up to 64 feet in the upper reservoir. An overview of the project is shown in Figure 4.3-18.

Figure 4.3-20 Seneca Pumped-Storage Project (courtesy of MWH)

Paving of the Seneca Upper Reservoir began in September 1967 and was completed in September 1968. During first filling, settlement of the foundation overburdens and fill materials led to some cracking of the asphaltic-concrete floor lining. Repairs were made successfully and the reservoir put into operation in 1970. By the mid-1980's, annual shut downs to repair minor cracks in the floor prompted a complete overlay of the floor, completed in 1986. The asphaltic-concrete lining on the interior side slopes however was not replaced and continued to deteriorate over the next few years. The impervious asphaltic-concrete lining had become brittle and showed signs of aggregate raveling, cracking, slide creeping and delaminating between the upper and lower impervious asphaltic-concrete layers, especially in the zone of maximum reservoir fluctuation.

The condition of the liner was evaluated in detail to help identify conditions that led to the deterioration of the lining and to assist in identifying appropriate rehabilitation options. Being one of only a few pumped-storage projects in the United States with an asphaltic-concrete lined reservoir, there was little information on asphaltic-concrete used for this application. Core samples were taken from the reservoir floor and side slopes. A series of laboratory tests was performed on the core samples to determine properties of the in-place asphaltic-concrete lining. Tests were performed for specific gravity, density, gradation, penetration, and softening point in accordance with ASTM standards.

Based on the condition evaluation, the upper impervious layer on the side slopes was found ineffective as an impervious membrane. Despite the condition of the asphaltic-concrete lining, piezometer data and drainage system data indicated that the lining system was still providing an effective barrier against seepage.

The condition evaluation made it obvious that liner rehabilitation was needed. The general requirements for the rehabilitation included: watertightness, flexibility to accommodate movements under daily loading and unloading, durability to provide another 30 years of service life under the extreme exposure conditions to which it would be subjected, and economic to install and maintain.

Prospective lining materials considered for the rehabilitation included: asphaltic-concrete, Portland cement concrete and/or shotcrete, steel, clay, soil cement, and geomembranes. Through a screening study using the general requirements established for the project, a number of alternatives were eliminated. Portland cement concrete/shotcrete and soil cement were deemed too rigid, and steel plate was found uneconomic for such a large installation. Compacted clay would require the side slopes to be flattened to remain stable under reservoir drawdown, and there were insufficient quantities of suitable clay available locally.

As a result of the screening study, two alternatives were found to satisfy the criteria established and included: 1) replacement in kind with asphaltic-concrete, and 2) a geomembrane overlay. Construction methodologies, schedules, and cost estimates were developed for both alternatives for comparison purposes. The evaluation found the replacement-in-kind alternative to be more favorable overall. Schedule wise, there was no advantage to either method, as both rehabilitation options could be completed in one year with the proper resource allocation.

The performance of the geomembrane liner was found to be highly dependent upon the quality of seam welding and the long-term holding capacity of the anchors into the existing asphaltic-concrete lining, rather than the durability of the geomembrane itself. Given the condition of the existing asphaltic-concrete, the guarantee of long-term anchor capacity at every location was deemed questionable.

The performance of the original asphaltic-concrete liner system was also considered in the evaluation. The fact that the original asphaltic-concrete lining had performed commendably over its 30-year-life demonstrates the suitability of the replacement-in-kind alternative and the more forgiving nature of this type of construction.

The contractor selected for the work had extensive experience in paving and also had experience in paving on steep slopes. Techniques developed by the contractor for horizontal paving of high-speed automobile racetracks were adapted to the unique circumstances of the Seneca Upper Reservoir. The result was that over 1,400,000 square feet of asphaltic-concrete facing was placed on a steep slope in a 10-week window between the end of winter and the beginning of the summer peak energy demand season. Unique aspects of construction that helped achieve the challenging project goals included: using a jet engine that is normally used to dry racetrack surfaces for automobile racing to dry the reservoir surface before paving with the highly moisture-sensitive impermeable asphaltic-concrete mix, thereby extending the hours during which paving could be performed; and using a turntable system to rotate construction vehicles on the narrow reservoir crest road, reducing travel times and increasing productivity.

A strict construction quality control and quality assurance program was key to achieving the high quality of the constructed project. Leakage monitoring instruments verified that leakage from the entire 115-acre reservoir was virtually eliminated as a result of the new asphaltic-concrete lining and mastic-finishing layer. The potential for saturation of the reservoir embankments due to seepage or leakage was greatly reduced. The rehabilitation resulted in a much more reliable and safer project from a dam safety perspective, benefiting the residents of the Allegheny River Valley living downstream of the dam.

The rehabilitation is expected to give reliable and economic service for another 30 years, allowing the project to meet increasing energy demands in an environmentally friendly manner. The project will continue to contribute to the social and economic advancement of the community, as it has for the past 30 years, by providing employment to local individuals and organizations, access to public recreation at the Upper Reservoir site, as well as providing a source of clean, renewable energy for residential and commercial use throughout the region.

4.3.7 Collective Knowledge

1. Cavalcanti, Alberto Jorge C. Tavares *et al.* (2000). "Rehabilitation of a generating unit affected by alkali-aggregate reaction" in *11th International Conference on Alkali-Aggregate Reaction, Quebec City, QC, Canada, June 2000: Proceedings.* USCOLD, Denver, CO. p. 1253-1262.

2. Creager, W.P. & Justin, J.P. (1950). *Hydro-Electric Handbook*, 2nd edition. John Wiley and Sons, New York, NY.

3. Jansen, Robert B. (1988). *Advanced Dam Engineering for Design, Construction and Rehabilitation.* Van Nostrand Reinhold, New York, NY.

4. Perry, E.B. (1987). "Dispersive Clay Erosion at Grenada Dam, Mississippi," in *Engineering Aspects of Soil Erosion, Dispersive Clays and Loess.* [Geotechnical Special Publication No. 10]. C.W. Lovell & R.L. Wiltshire (eds.). ASCE, New York, NY. p. 30-45.

5. United States Army Corps of Engineers (USACE). (1995). *Engineering and Design - Gravity Dam Design.* [EM 1110-2-2200]. USACE, Washington, DC.

   http://www.usace.army.mil/publications/eng-manuals/em1110-2-2200/tl.pdf

4.3.8 Technical References

American Electric Power (AEP). (1997). Project files for American Electric Power, Columbus, OH.

American Electric Power (AEP). (1999). Project files for American Electric Power, Columbus, OH.

American Society of Civil Engineers (ASCE). (1992). *Guidelines for Rehabilitation of Civil Works of Hydroelectric Plants.* ASCE, New York, NY.

American Society of Civil Engineers (ASCE). (1995). *Guidelines for Design of Intake for Hydroelectric Plants.* ASCE, New York, NY.

Blanchette, D.G., Hartford, A., Elliott, M.A., Koseatac, A.S., (1997). Centennial Mill Dam Major Rebuild. ASDSO Annual Conference.

Bliss, M. (2003). "Jet Grouting for Seismic Remediation" in *USSD Newsletter,* July 2003, Issue No. 130.

Brookfield Power New York. (1989). Project files for Brookfield Power New York, Liverpool, NY.

Brookfield Power New York. (1997). Project Files. Brookfield Power New York, Liverpool, NY.

Dridge, R.G. & Mathews, D.L. (1997). "Outlet Channel Repair Milford Dam, Kansas". (Authors affiliated with USACE).

Findlay, R.C. & Rabasca, S.J. (2003). "Diversion Dam – Seismic Stability Assessment and Rehabilitation" in *Waterpower XIII: Advancing Technology for Sustainable Energy, July 29-31, 2003, Buffalo, New York, USA. Technical Papers.* [CD-Rom]. Kansas, MO.: HCI, 4p.

Forrest, M., Callan, S., Feldsher, T., Mejia, L., Lawton, G., Ogren, P. (2004). *Stone Column Construction Stabilizes Liquefiable Foundation at Lopez Dam,* [Excerpts from ASDSO Annual Conference]

Greenman, J.H. *et al.* (1997). "Wallowa Falls Hydroelectric Project - Dam Rebuild," in *Seventeenth USCOLD Annual Meeting, April 7-11, 1997, San Diego, California.* USCOLD, Denver, CO.

Hartford, D. (1998). "BC Hydro" in *Hydro Review.* HCI Publications Inc.

Kleinschmidt, (1992). Project files for Farmington River Company, Pittsfield, ME.

Kleinschmidt, (1998). Project files for American Electric Power. Columbia, SC.

Lawton, G., Forrest, M., Mejia, L., Feldsher, T., O'Halloran, D. (2004). *First Use of Stone Columns in California Under State Regulatory Jurisdiction – The Seismic Remediation Design of Lopez Dam,* [Excerpts from ASDSO Annual Conference]

Manguarian, D. (1994). "Ingenuity Saves Dam" in Project files for International Development Bank, Washington, DC.

Meisenheimer, J.K. & Wagener, C.D. (1997). "Developing a Management Program for Concrete Growth from Alkali-Aggregate Reaction in Concrete Dams" in *Waterpower '97: Proceedings of the International Conference on Hydropower held in Atlanta, Georgia, Aug. 5-8, 1997.* ASCE, New York, NY. p. 274-284.

MWH. (1988). Project files for Wisconsin Electric Power Company. MWH, Chicago, IL.

MWH. (1995). Project files for Rosebud Sioux Tribe, SD. MWH, Chicago, IL.

MWH. (1997). Project files for Los Angeles County Department of Public Works, CA. MWH, Chicago, IL.

MWH. (1999). Project files for Los Angeles County Department of Public Works, CA. MWH, Chicago, IL.

MWH. (1999). Project files for Safe Harbor Water Power Corporation. MWH, Chicago, IL.

MWH. (2000). Project files for Sacramento Municipal Utility District (SMUD), CA. MWH, Chicago, IL.

MWH. (2000). Project files for City of New York, NY. MWH, Chicago, IL.

MWH. (2001). Project files for City of Decatur, IL. MWH, Chicago, IL.

MWH. (2001). Project files for Southern California Edison, CA. MWH, Chicago, IL.

MWH. (2001). Project files for Upper Peninsula Power Company (UPPCO), MI. MWH, Chicago, IL.

MWH. (2005). Project files for Miami Conservancy District, OH. MWH, Chicago, IL.

Prendergast, J. (1991). "Historic Dam Rehab: Handle With Care" in *Civil Engineering,* Vol. 61, No. 10, October, p. 50-53.

Rabasca, S.J. & Findlay, R.C. (2003). "Seismic and Seepage Remediation of Diversion Dam" in *Proceedings of the 2003 USSD Annual Meeting and Conference, Charleston, SC.* USSD, Denver, CO.

Rabasca, S.J. *et al.* (1999). "Comparison of Borehold Advancement Techniques on SPT N-Values" in *Proceedings of the Nineteenth Annual USCOLD Lecture Series, Atlanta, GA.* USCOLD, Denver, CO.

Seed, B.H. *et al.* (1984). "Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations" in *Journal of Geotechnical Engineering,* Vol. III, No. 12, December 1985.

United States Army Corps of Engineers, Portland District. (USACE). (1975). "Harlan Dam Remedial Modifications", in *American Society of Civil Engineers, Civil Engineering,* August.

Utah State Department of Water Resources (UTDWR). (2003). UTWDR website. http:// www.water.utah.gov/

4.4 Powerhouses

4.4.1 Function

The primary purpose of the powerhouse is to house and support the electrical and mechanical equipment associated with the generation of power, and to transfer the static and dynamic loads of the equipment, as well as the hydraulic loads, to the foundation of the structure. In addition, the powerhouse protects the equipment from being damaged by weather, animals, vandals, etc. Powerhouses usually provide space for control equipment, batteries, oil storage, heating/ventilation equipment, and drainage/dewatering pumps. The powerhouse may also include, in some instances, a service area where the maintenance and repair of the equipment can occur, facilities to accommodate personnel involved in the day-to-day operation and maintenance of the plant, and facilities for visitors to tour the facility.

There are several different types of powerhouses. The first type is a fully enclosed powerhouse (e.g. underground powerhouses, chambers within dams and underground powerhouses constructed in rock chambers) where the generators and control room are located inside a building. The second type is partially open, with the generators located on the roof, and the controls and auxiliary equipment located inside the building. The third type is a fully open powerhouse where the generator and all the equipment are located outside in weather-proof enclosures. The powerhouse houses the turbines, generators, auxiliary generating equipment and controls. For some projects the powerhouse is a water retaining structure and is part of the dam. At other projects the powerhouse is a separate structure located downstream of the dam.

The components of the powerhouse are the roof, the structural framing and walls, the main generator floor, the turbine floor level, and draft tube level. Often, an overhead bridge crane or a gantry crane is included for performing unit overhauls and other maintenance functions. The roof construction can range from timber framing with timber planks covered with asphalt shingles or clay tiles, to steel framing with corrugated metal roofing or rock surfaces, to cast-in-place concrete or pre-stressed pre-cast concrete panels.

The walls can be constructed from wood, masonry, concrete, rock, pre-cast concrete panels or steel framing with metal siding. When an overhead crane in incorporated into the powerhouse, the walls of fully enclosed powerhouses typically support the overhead bridge crane. The partially open and fully open powerhouses generally have a gantry crane or require a mobile crane for performing maintenance and repair activities.

4.4.2 Problems

This section discusses some of the more common powerhouse problems in the following areas:

a) Foundation.
b) Stability.
c) Structural Damage.
d) Seepage.
e) Roof Damage.
f) Draft Tube Damage.

a) Foundation

Common foundation problems that powerhouses experience include loss of support, differential settlement, and heave. Loss of support is a condition where the foundation material has lost some of its original strength, or has eroded, causing the structure to settle and/or tip, potentially causing damage to the structure and equipment, and misalignment problems with rotating equipment such as the turbine and generator. This can be caused by a low strength material beneath the structure that compresses when the structure loads are applied. It may also be due to the foundation material, such as limestone, till or soil, being dissolved or eroded by groundwater or displaced by subsurface flows. Differential settlement occurs when the foundation material strength varies beneath the structure, such as when one end of structure is founded on rock or low compressible soil, and other end is founded on a weaker rock or more compressible soil, respectively. Differential settlement can cause alignment problems for the power generating equipment or can result in severe damage to penstocks, generating units, and the powerhouse structure. Heave occurs when the foundation expands causing the structure to rise up. This condition can occur when expansive soil exists beneath the structure or when excess pore water pressure occurs beneath structure.

b) Stability

A common problem experienced at older plants concerns the stability of the structure. Stability problems are generally associated with powerhouses acting either as water retaining structures (e.g. when the powerhouse is integral with the dam) or supporting embankments (e.g. when the powerhouse abuts and supports an embankment). This concern may be the result of a change in design loading on the structure, such as recent earthquake events, that results in a change to the seismic criteria at the site. For example, several hydroelectric sites in the eastern half of the United States have been required by the FERC to be reassessed for seismic stability by superimposing west coast earthquake conditions beneath east coast hydroelectric structures. Despite the relatively infrequent occurrence of significant seismic events in the eastern part of the country, the assumption is that there is no reason why a seismic event of similar duration and magnitude to a west coast event could not occur on the east coast. Where this seismic reanalysis has taken place, the end result has been to increase the seismic forces assumed to act on existing hydroelectric structures as compared to the original design basis. Powerhouses that do not act as water retaining structures typically were not designed for seismic loading.

Another design change that impacts the powerhouse stability is a change in design assumptions regarding the design flood event. Many older hydroelectric facilities either were not analyzed for the design flood event or, if they were, the design criteria used for the analysis yielded significantly lower values than today's criteria. For example, one series of hydroelectric facilities that were built in the mid 1950's were designed for a flood event of approximately 20,000 cfs, corresponding to what was estimated to be the 100-year flood at the time. The flood assessment for these facilities was updated in the mid 1990's under the FERC's High Hazard criteria which determined the design flood was the Probable Maximum Flood (PMF), estimated at approximately 70,000 cfs. This change of design criteria required the owner to reanalyze all of the water retaining structures and to reassess the potential of flooding critical facility components, including the powerhouse, due to higher than originally designed impoundments and tailrace water elevations. Another example of changing design criteria is in the area of uplift assumptions. Some design references from the 1930's typically recommended taking a 50% reduction in the foundation uplift pressures when performing stability analysis when foundation grouting was performed. Today this assumption has been found to yield very un-conservative and inaccurate results and many owners have had to install rock anchors to compensate for these.

Changes in project operation can result in the powerhouse structure no longer meeting current stability requirements. One example of this is raising of the impoundment elevation to increase power generation and water storage. Where the powerhouse is integral with the dam, the higher impoundment elevation will cause a direct increase to uplift and hydrostatic forces acting on the structure. Where the powerhouse is located downstream of the dam, there is the possibility that an increase in the impoundment elevation will cause increased seepage pressures through, or beneath, the dam that can have a destabilizing effect on the powerhouse.

c) Structural Damage

Older powerhouses can exhibit a wide variety of structural problems. They include deterioration in concrete, masonry, timber, or steel members, overstressing of critical structural load carrying members, lack of maintenance, freeze-thaw damage, erosion, and abrasion, to name a few. The following are some examples of the more typical structural problems in powerhouses.

Concrete, especially in the colder northern climates can be prone to deterioration. Deicing salts are notorious for the deteriorating effects they have on concrete. Where the powerhouse is large enough for crew trucks to enter during the wintertime, the snow and salts will melt off the parked vehicle, causing the concrete slab and columns supporting the vehicles to deteriorate over time. This damage is very similar to what occurs in municipal parking garages. Expansive concrete, known as alkali aggregate reactivity (or AAR) occurs when the alkali content of the cement and aggregate are high, and when sufficient moisture is present to sustain a chemical reaction. The chemical process is relatively slow, often taking several years, and will result in a continuous expansion and cracking of the concrete. More discussion on this can be located in the section on dams found earlier in this chapter.

Structural columns and beams, whether they are steel, wood or concrete, can become overstressed for a number of reasons. Where there is no powerhouse crane, or where the crane is not as useful, beams and columns become popular locations for fastening hoists and other temporary lifting devices useful during overhauling a turbine and generator. These temporary connections are often not engineered and the structural components become overloaded and deformed. Another example is on powerhouses where newer ballasted roof systems have been installed. In many cases the roof beams/columns were not designed to support the added load from the roof ballast. Coupled with high snow loads in the wintertime, isolated or catastrophic failure of structural supports can occur.

Some older powerhouses have their origin as former mill facilities. These mills were located on rivers to make use of the waterpower to run hydro-mechanical apparatus, such as sawmills, or to make paper, flour, or linen. When these industries ran into financial difficulties, particularly in the 1920's-1930's, many of these mills were converted to more profitable hydroelectric generation facilities. From a civil works perspective, the challenge with these facilities was to assure the structural supports within the converted powerhouse were adequate for the intended loads. One example was a mill that was built around 1900. The turbines were originally connected to overhead pulleys and operated a wide variety of belt driven machines. This facility was converted to a hydroelectric facility in the 1930's. In the 1970's the owner wanted to better utilize the large storage area in the back of the powerhouse to store large equipment such as spare turbine components and electrical components. The owner had two options to transport equipment to the rear of the building. The first option was to drive crew trucks through the large roller doors in the front of the building and traverse the 150 feet to the rear of the building for loading and unloading. This required an engineering investigation of the main powerhouse floor that consisted of a concrete supporting slab and piers that had not been modified since the building was originally constructed. The slab was found to possess insufficient steel reinforcing and would require several hundreds of thousands of dollars of reinforcement to support vehicular loads. The other option was to use the original overhead crane to transport the equipment from the front of the powerhouse to the back storage area. Given its age, an engineering assessment of the crane was performed, (including a structural assessment to include what would be needed to motorize the bridge drive and hoist). An assessment was also performed of the runway supports, as these were the original lattice type columns popular with that vintage of mill construction. Based on cost, the owner chose to motorize the existing crane and to reinforce the existing runway and columns along with reinforcing selected column piers beneath the powerhouse floor. This example also illustrates some of the difficulties encountered when analyzing older structural components using modern day criteria. For example the runway support columns, when analyzed for the increased crane load, showed that some modifications were required just to support the historical column loads without even considering the additional crane loading, yet the building had been standing and in use for over 70 years.

d) Seepage

A common problem in powerhouses is water seeping into the powerhouse. Minor seepage is not typically a serious problem because it can be easily collected and drained. On the other hand, significant amounts of seepage can be a serious problem, and if solutions are not found the plant may become flooded or equipment damaged. Excessive moisture in the plant can also cause rising humidity leading to further corrosion of important equipment. Sources of seepage can include a failed waterstop within the concrete, an excessively high external pore water pressure against the buried portions of the powerhouse foundation or leakage from the penstock or pipelines. In one example, a new powerhouse had been constructed and was undergoing unit commissioning. A significant amount of leakage was evident through the below grade portion of the upstream wall of the powerhouse. It was also observed that when the new six-foot diameter penstock was dewatered, the seepage through the powerhouse stopped. Upon investigation, it was determined that the penstock was fitted with a tap that was improperly sealed, causing the backfill behind the powerhouse to become fully saturated which resulted in seepage through the powerhouse wall. The penstock was dewatered and the tap was sealed. Another example was a powerhouse located at the toe of an earth dam. Seepage through the earth dam caused the pore water pressure behind the powerhouse to build up, causing significant seepage through the powerhouse wall. This problem was corrected by installing a french drain behind the powerhouse allowing the seepage through the dam to be collected and effectively drained off, and keeping the pore water pressure behind the powerhouse within acceptable limits.

Seepage through concrete can also be caused by porous concrete. Construction techniques in the 1910's to 1930's commonly used, what was called at the time, a "3 bag mix", or three 95 lbs bags of cement per cubic yard of concrete. Compared to today's concrete mix, where typically six or more bags of cement are used per cubic yard of concrete, this ratio resulted in a very lean concrete mix. Experience with these older concrete structures has taught that concrete made with the "3 bag mix" is generally of lower strength, more susceptible to freeze-thaw, and becomes more porous after several years of increasing seepage which results in more seepage compared to today's conventional mixes.

Seepage can also be caused by inoperable internal drainage systems. Powerhouses and dams are typically built with internal drainage systems designed to collect water that seeps through structures. When these systems become inoperable, water will take the path of least resistance and exit in areas of the powerhouse that are unsafe (e.g. in and around energized electrical equipment), or in ways that can result in unstable structural components (e.g. behind walls that were never designed to support excessive pore water pressures).

e) Roof Damage

Roof damage can be caused by old age, poor maintenance, wind, and ice. Failure of any roofing system will ultimately lead to damage to the powerhouse equipment below and, if undetected for a long period, can result in deterioration to structural components (i.e. rusting of steel members or rotting of timber members) that support the roof. Damage to roofing systems can also be caused by excessive traffic (i.e. people and equipment) that occurs on roofs from time to time. Improper roof drainage can result in ice formations that can tear and rip roofing components. High winds can start to peal corners of single ply roofs that will cascade to entire sections of roofs being blown off.

f) Draft Tube Damage

Some draft tubes are built integrally with the powerhouse substructure. These draft tubes consist of either formed concrete or concrete encased steel liners. These draft tubes can become damaged for several reasons. Corrosion to the steel liner can occur exposing the concrete behind the liner to the erosive forces of water exiting the turbine. Because the concrete is not as durable as steel, once the steel liner is gone, large voids can form very quickly behind the liner causing a significant loss of concrete. On draft tubes that do not have steel liners, corrosion of reinforcing steel can cause concrete to spall. Once the spalling begins, the hydraulic smoothness of the draft tube is compromised, causing accelerated erosion of the concrete in and around the original spalled area. Without repair, this can result in a significant loss of concrete over a short period of time along with diminished turbine performance. In addition to affected power output from the turbine, excessive cavitation can also result in destructive forces on the draft tube. Cavitation has been known to destroy steel liners and concrete forming the draft tube. In instances where there is poor concrete consolidation behind the steel liner, vibration of the steel liner can result in partial or complete failure of the liner, resulting in damage to the powerhouse substructure. Draft tube elbows can also be susceptible to concrete erosion due to the high velocities and hydraulic forces that occur at that location. Vertical draft tube systems that allow the water exiting the draft tube to impinge directly upon the powerhouse foundation material should be examined closely for erosion of the foundation material resulting from the high velocities impinging on the foundation material. Where draft tubes are hung below the turbine and are not integral with the powerhouse substructure, failure of the lateral supports that brace the draft tube to the powerhouse substructure can occur. These lateral supports are typically subject to wet/dry conditions and turbulence that can cause premature failure of these braces.

4.4.3 Corrective Measures

For the problems identified in Section 4.4.2, the following are possible corrective measures.

a) Foundations

Problems with powerhouse foundations can be repaired using a number of techniques. Foundation grouting can consist of injecting either cement or chemicals into the foundation beneath the powerhouse. The benefits of grouting include reducing seepage, filling voids and strengthening weak foundations. Where the voids are particularly large, mass concrete can be used in lieu of grout. Rock/soil anchors are useful for strengthening walls to resist external water and soil pressure, or resisting uplift pressures underneath structures. Conventional piles (concrete, steel, timber) can be used to support structures that sit on weak soils. Foundation compaction methods such as vibro compaction and dynamic compaction are effective methods to increase the bearing capacity of foundations and reducing settlement. Adding mass concrete can also be a solution where additional weight is required to support an instability problem.

b) Stability

A change of design criteria in the areas of PMF and seismic analysis can be a costly issue for an owner to deal with. Current state of the art criteria generally employ more accurate assumptions (for example, better hydrologic data or seismic understanding) compared to the original design, and are intended to better model the true stability of the structure. While there is not necessarily any single prescribed measure that should be taken when the use of current design criteria show the powerhouse to be theoretically unstable, it is recommended that owners take an active role in determining what revised criteria should be applied, and their appropriate application, prior to proposing, investigating or implementing any remediation. It is recommended that owners work closely with the design engineer and dam safety regulators to discuss the updated criteria and to determine their appropriate application to the specific structures.

Proposed changes in the project operation should always involve a thorough review of impacts to related and adjacent civil works. As in the previous example where the impoundment was raised to increase generation and storage, impacts to the other civil works at the project should be thoroughly considered in order to make an informed decision on how to proceed. For example, if the benefit of raising the impoundment was \$1 million/year, the owner should also consider what costs are necessary to strengthen the dam against the additional uplift, hydraulic and other forces resulting from the higher impoundment, costs to address the increased likelihood of flooding the powerhouse, and the cost of flooding any upstream property owners.

c) Structural Damage

Structural problems within the powerhouse involve a wide variety of problems, solutions and repairs that are so broad that a thorough discussion is outside the purpose of this document. Options include:

- Concrete repair techniques.
- Grouting.
- Steel repair and reinforcement techniques.
- Timber repair techniques.

The use of licensed engineering professionals identify to inspect, perform analysis, recommend repairs, and oversee the repair process is recommended to assure the appropriate technique is properly applied to correct the problem. It is also recommended that a thorough annual inspection be made of the powerhouse structure, substructure and related components. This will help identify problems before they become serious, and also reduce the scope and cost to make necessary repairs. Also, prior to correcting a problem, remember to consider the possible impact of future upgrades or changes. This will help to avoid the situation as it occurred at one site. Only a year after completing a major repair to the powerhouse floor supports, the need arose to bring in bigger trucks, requiring the strengthening, of the very same floor components, just repaired.

d) Seepage

Seepage problems are one of the easiest to spot and sometimes one of the most costly to repair. Owners should thoroughly evaluate the cause of the seepage to determine if it should be corrected because not all seepage needs to be corrected. In fact in some cases stopping the seepage can actually cause an increase in pressure behind the exit point, resulting in structural stresses that may not have been considered in the original design. For example, a powerhouse was located at the base of a vertical rock cliff that was several hundred feet tall. Seepage through the rock for a number of years was causing water to collect behind the powerhouse. The owner initiated a grouting program of the rock cliff face. Some time after the grouting was complete the pore pressure built up within the rock face causing the entire rock face to collapse, destroying the powerhouse. In this example, grouting effectively stopped the flow of water, but there was no method to accommodate the build-up of pore water pressure, which resulted in catastrophic collapse.

One of the most effective means of dealing with seepage is to locate the source of water and stop it where it originates. When this is not possible, accommodating the seepage by channeling it so as not to affect project operation should be considered. Where the seepage has a negative impact on project operations, techniques such as grouting are commonly employed. Grout injection using either cement or chemical grout can be effective. One type of chemical grout is a polyurethane hydrophobic material that uses water to react with to form a material resembling foam. This grout remains flexible and can be used in moving cracks. Other grouting products can provide satisfactory results when properly applied and should be thoroughly researched before implementing. One note about grouting programs is that they are seldom successful at stopping the leakage at the source. When they are successful at stopping leakage, there is a risk that the leakage will be relocated through a different path that may be better or worse than the original. For this reason, when the leakage is not excessive, the preferred option is to collect the leakage at the exit point and route it in a safe manner. This will also provide the ability to monitor the leakage over a period of time.

When the source of leakage is known, installing an impervious material over the entrance can be a viable option. The impervious material can include plastic sheets, cinders, or other material designed to prevent water from entering the seepage path. Where leakage is determined to be caused by a malfunctioning drainage collection system, efforts should be made to clear the drainage system from whatever obstruction is preventing the leakage from freely entering the system.

e) Roof Damage

There are a number of references that discuss roof repair options and techniques. They generally depend on the make and manufacturer of the roofing system, and the manufacturer should be consulted when contemplating a specific repair.

f) Draft Tube Damage

Damage to draft tubes is typically repaired using general concrete or steel repair techniques. Deteriorated steel liners can be repaired with steel patches. Pre-heat and non-destructive examination (NDE) requirements may also impact on how the steel repair is performed. Concrete deterioration to draft tubes can be repaired using any number of high strength concrete repair products on the market. Of particular emphasis is the need to perform periodic inspections of the draft tube and related supports. Due to the high velocities within the draft tube, damage can progress at a very fast rate. Since the draft tube is typically inaccessible for normal inspections, periodic inspections using a diver or remote operated vehicle (ROV) is recommended. Depending on the age of the turbine/draft tube, the inspection interval could be anywhere from 1 to 5 years. If repair is determined necessary as a result of an inspection, a root cause evaluation would be helpful in determining the cause of the deterioration and addressing corrective measures, thus avoiding the problem from occurring again.

4.4.4 Case Histories

No. 1 Deteriorated Concrete Spiral Case
Wapato Indian Irrigation Project (USBR, 2000)

The Drop #3 Powerhouse of the Wapato Indian Irrigation Project was originally constructed in 1933. A feature of this structure is the unlined concrete spiral case surrounding the turbine runner. Over the years cracks have developed in the downstream wall of the spiral case and structure. During operation, water pours out of these cracks, and has caused some corrosion of the reinforcement steel in the concrete. To correct the problem a waterproof coating was applied the inside walls of the spiral case to prevent water from entering the cracks in the concrete.

No. 2 Powerhouse Instability (Flood Loads)
Buck Hydroelectric Project (AEP, 1993)

The Buck Hydroelectric Project located on the New River in Virginia had inadequate factors of safety for the PMF loading requiring the powerhouse to be stabilized. The powerhouse is approximately 170 feet long and houses three vertical, Francis type turbines. The powerhouse was stabilized using post tension anchors installed vertically on the upstream side through the intake piers. There are a total of seven anchors installed ranging in size from 735 kips to 945 kips.

No. 3 Powerhouse Instability (Uplift Pressures)
Schaghticoke Project (Brookfield Power New York, 1999)

The Schaghticoke Hydroelectric project, constructed on the Hoosic River in New York, consists of a 1,700 foot long rock cut canal, and a 850 foot long 12 foot diameter pipeline in the form of an inverted siphon which spans the Hoosic River valley to a 40 foot diameter surge tank. Four 6-foot diameter penstocks connect the surge tank directly to the four turbines in the powerhouse. The gross head of the site is 150 feet. The facility was built in 1908.

Originally, replacement of the four penstocks leading to the powerhouse was being considered for life extension purposes. During the data-gathering phase of the conceptual design, it was determined that the powerhouse was founded on glacial till. The subsurface investigation determined that two aquifers existed beneath the powerhouse. The resulting uplift from these aquifers affected the stability of the slope supporting the four penstocks and the stability of the powerhouse. It was also determined that surface water from the adjacent slope would cause surficial slope stability problems.

An additional problem was the control of water levels in the area upstream of the powerhouse to allow excavation and construction in the dry.

Prior to construction of excavation for the penstock installation (see Case History in Chapter 5), a series of relief wells were installed upstream of the powerhouse. During construction these wells were utilized to dewater the excavation area by installing high capacity pumps. For permanent reduction of the uplift forces, the relief wells were fitted with a permanent gravity collection system. To remove surface water from runoff or snow melt, the penstock slope was constructed with several bench cuts and an under drain system to intercept and collect surface water and a conveyance system to discharge the collected water outside of the penstock slope.

No. 4 Life Extension Study
Cabot Project (Kleinschmidt Associates, 1993)

The Cabot Hydroelectric Project is located in Turners Falls, MA, and consists of a dam, an 11,600-foot long power canal, and two powerhouses generating 57 MW with eleven units at 15,000 cfs. The initial powerhouse (five units, 5.6 MW) was constructed in 1920 on an existing canal, and in 1935 the canal was extended and a second powerhouse added (six units, 51 MW). The concrete gravity dam is constructed in a series of independent sections, separated by rock outcroppings. The original dam had been rebuilt or replaced in sections since 1935, with the most recent section replaced in 1968.

In the early 1990s the project's FERC license was to expire, and the owner required a Life Extension Study to evaluate upgrades or major modifications as may be required for the hydroelectric project to allow operations to continue through the end of the new 50-year license. The study was to assess upgrade needs for all civil, mechanical, electrical, hydraulic, and hydro-mechanical systems associated with the power canal and the powerhouses. In addition, the study was to determine the cause of major cracking and leakage in the 1935 powerhouse. The scope of the study was open to allow the consultants performing the work to assess conditions based on findings.

The study evaluated the addition of a third powerhouse and / or increasing the hydraulic capacity of the 1935 turbines. Hydraulic models (computer) of a proposed increase in hydraulic capacity indicated a canal surge problem during a load rejection. The canal included a gated spillway in the forebay of the 1935 powerhouse, but the gates were rack and pinion operated and not intended for use in suppressing canal surges. The sixteen gates (ten feet high) in the canal intake structure were screw stem operated and could not be closed fast enough to stop the flow into the canal. A functional load rejection test of the 1935 powerhouse was performed to verify the model's findings. The test indicated that if four or more of the existing 1935 turbines were to undergo a load rejection, a canal surge would occur with potential detrimental impacts. While the functional test confirmed the model results at the forebay, the test indicated that the most significant surge would occur immediately downstream of the canal intake structure and impact numerous industrial buildings.

To mitigate the canal surges, the spillway gates in the powerhouse forebay were mechanized and automated to provide an outlet for the surge wave. The canal intake gates were automated and upgraded with hydraulic cylinders to reduce the time for closure reducing inflow to the canal and surge problems at the intake. In addition, the canal gate operating procedures were modified to minimize the height of gate opening needed to maintain flow into the canal. Modifications also included four means of power supply to all gates (2 primary, 2 backup).

Economic studies indicated that adding a third powerhouse or increasing the hydraulic capacity of the 1935 turbines was not economically feasible. The study concluded that the existing water-cooled transformers should be replaced and the power distribution equipment upgraded. If the plant was to be automated and operated remotely, all electrical switchgear and hydro-mechanical control systems (water actuated servo cylinders) should be replaced. As a result of the upgrading of the power distribution, electrical, and hydro-mechanical systems, it was determined that the generators could be economically rewound and upgraded to provide more power output, as the existing equipment was generator limited.

The turbine spiral cases were unlined, and the soft weak concrete in the 1935 powerhouse was believed to be the cause of the leakage through the generator floor. The life extension study determined that the leakage was exacerbated by the pressure grouting of the floor that had be done in the 1960s and 1970s to reduce the leakage. The study further determined that the real cause of the leakage was the plugging of a drainage system that was constructed over the spiral cases and two feet below the finished floor. The plugging occurred in the early 1960's when a coal fired boiler house attached to the hydroelectric plant was demolished, and the drain outlets plugged. The subsequent grouting programs had filled the drainage system, and also resulted in the generator leadings being grouted into their conduits. The study proposed reducing the leakage though the floor by grouting of the vertical construction joints (using hydrophobic polyurethane) and the few cracks observed in the turbine spiral cases.

The study determined that the significant structural cracks in the downstream side of the 1935 powerhouse occurred in the 1940s when a flood raised the powerhouse tail water to the level of the generator floor. The cracks developed as a result of pressure buildup in a chamber over the discharge end of the draft tubes. The pressure was caused by the surging water levels in the chamber and the absence of functioning air vents. Large grated openings in the floors of the chamber had been permanently plugged intentionally, and three-inch diameter air vent holes in the vertical walls had been plugged by wasps and barn swallows. While the crack was determined to not impact the structural integrity of the powerhouse, the associated open gaps and monitoring were considered to be an expense. Repairs consisted of increasing the vent holes in the walls to 8-10 inches in diameter, and sealing the crack with an elastomeric material.

No. 5 Antiquated Bridge Crane Bonneville No. 1 Powerhouse (USACE, 2000)

Bonneville #1 Powerhouse has been producing electrical power for the Pacific Northwest since 1933. The powerhouse bridge cranes have been used since that time for annual maintenance and 5-year overhauls at each of the 10 main turbine/generators, and small station service unit, as well as routine powerhouse operations. During the past six years, the first powerhouse has been undergoing a major rehabilitation. Each of the main unit turbines is being replaced, and five of the generators are being rewound.

This program resulted in increased use of the powerhouse bridge cranes by both project personnel and the rehabilitation contractor and has highlighted problems with some of the crane components. The controls were erratic and sometimes non-responsive. The trolley rails and wheels were showing excessive wear (spalling of the rails and grooves in the wheels). The cranes were skewing during travel, occasionally to the point that one would stop traveling and the second was used to dislodge the first. The bridge rails were wearing (spalling of rail and corresponding wheel damage) and not within Crane Manufacturers Association of America (CMAA) alignment criteria. The wooden sleeper under the rails was deteriorating and some of the rail clip bolts were corroded almost completely away.

The owner, the USACE, was faced with a project feature at the end of its service life, safety issues, and the need to keep the project powerhouse bridge cranes serviceable, both for performing the rehabilitation and afterwards for O&M.

A performance contract was written and awarded to replace mechanical items such as gearboxes, couplings, wheels, bearings, and shafting. Electrical components including motors, brakes, controls, wiring, conductor systems, and circuit breakers were also replaced. Structural modifications included replacing the operator cabs and the crane rails. Asbestos abatement and lead-based paint were issues that were also addressed.

As a result of the Bonneville #1 Powerhouse Cranes Project, life extension is now commensurate with the overall rehabilitation of the 70-year old power station.

No. 6 Deteriorated Supports and Antiquated Bridge Crane Minidoka Project (USBR, 1992)

The Minidoka Power plant near Minidoka, Idaho, was constructed about 1910. The original crane was an overhead bridge crane with manually operated chain drive for traversing and a manually operated hoist. On the upstream side the crane rail was supported by a steel girder and columns and on the downstream side the crane rail was supported by timber cribbing on top of a reinforced concrete girder.

Over the years, the timber cribbing had crushed at various locations along the length of the plant, creating peaks and valleys in the crane rail, which made the manual operation difficult.

The Owner, the Bureau of Reclamation, was planning a generator upgrade that would extend the life of the power plant at least 30 years and needed to use the bridge crane extensively for this upgrade.

As part of the generating unit replacement, the existing crane and timber cribbing was removed. A new crane rail and grout pad were installed on the downstream girder, the upstream crane rail was cut at both ends to add crane bumpers, a new electric crane was installed with pendant controls, and power conductors were installed on the downstream superstructure wall to provide power to the new crane.

As a result, the powerhouse cranes were upgraded and used for the generator rehab at Minidoka and the life extension is now commensurate with the overall rehabilitation of the 65-year old power station.

4.4.5 Collective Knowledge

1. American Society of Civil Engineers (ASCE). (1989). *ASCE/EPRI Civil Engineering Guidelines for Planning and Designing Hydroelectric Developments Volumes 1-4.* ASCE, New York, NY.
2. American Society of Civil Engineers (ASCE). (1992). *Guidelines for Rehabilitation of Civil Works of Hydroelectric Plants.* ASCE, New York, NY.
3. Brown, R.W. (2000). 2nd. Ed. *Practical Foundation Engineering Handbook.* McGraw-Hill Inc., New York, NY.
4. Creager, W.P. & Justin, J.P. (1950). 2nd edition. *Hydro-Electric Handbook.* John Wiley and Sons, New York, NY.
5. Electric Power Research Institute (EPRI). (1989). H*ydropower Plant Modernization Guide, Volumes 1-3.* Electric Power Research Institute, Palo, Alto, CA.
6. Federal Energy Regulatory Commission (FERC). (1991). *Engineering Guidelines for the Evaluation of Hydropower Projects.* FERC, Washington, DC.
7. National Association of Corrosion Engineers (NACE.), Houston, TX.
8. National Fire Protection Association (NFPA). (2000). *Recommended Practice for Fire Protection for Hydroelectric Generating Plants – 2000 Edition.* [NFPA 851]., NFPA, Quincy, MA.
9. Society for Protective Coatings (SSPC), Pittsburgh, PA.
10. United States Army Corps of Engineers (USACE). (1995). *Engineering and Design - Evaluation and Repairs to Concrete Structures.* [EM 1110-2-2002]. United States Army Corp of Engineers, Hyattsville, MD. http://www.usace.army.mil/publications/eng-manuals/em1110-2-2002/toc.htm

11. United States Army Corps of Engineers (USACE). (1995). *Engineering and Design - Planning and Design of Hydroelectric Power Plant Structures.* [EM-1110-2-3001]. United States Army Corp of Engineers, Hyattsville, MD. http://www.usace.army.mil/publications/eng-manuals/em1110-2-3001/toc.htm

4.4.6 Technical References

American Electric Power (AEP). (1993). Project files for American Electric Power, Columbus OH.

Brookfield Power New York. (1999). Project files for Brookfield Power New York, Liverpool, NY.

Kleinschmidt. (1993). Personal Communications, Kleinschmidt Associates, Pittsfield, ME.

United States Army Corps of Engineers (USACE). (2000). Project files for USACE, Portland District, Portland, OR.

United States Bureau of Reclamation (USBR). (1992). Project files for USBR, Technical Services Center, Denver, CO.

United States Bureau of Reclamation (USBR). (2000). Project files for the United States Bureau of Reclamation, Technical Services Center, Denver, CO.

4.5 Fish Passage Facilities

4.5.1 Function

The purpose of a fish passage facility is to allow migratory fish to pass around a physical blockage created by a hydroelectric project so that they may complete their reproductive cycle. Migratory fish spawn and/or rear in the habitat upstream of dams, with young fish (in the case of anadromous and resident fish) and grown fish (in the case of catadromous fish) returning downstream to complete their reproductive cycle.

The life cycle of diadromous fish species includes obligatory movement between marine and freshwater ecosystems. Catadromous species, such as eels, migrate downstream as adults to reproduce in the ocean. Anadromous species, such as American shad, river herring, salmon and sturgeon migrate from the ocean into rivers to spawn. Native or resident fish undertake migrations between their feeding and resting habitats, or inhabit different parts of a river that offer specific conditions that satisfy life cycle requirements of their different developmental stages.

Dams and activities associated with power generation confuse migrating fish by changing the flow release patterns in the projects tailwaters and/or blocking their movements, both actions resulting in the delaying of fish migrations. The water retaining structures themselves are physical barriers to the migration of adult and juvenile fish; and through the diversion of flow for power generation, hydroelectric projects may result in the alteration of flow within, or dewatering of a channel, stream, or river that fish would otherwise use during their migration.

When upstream migrating fish find their natural passages blocked, they then spend time and energy trying to locate alternate routes around the blockage, including finding an entrance to a fish passage facility. After getting past a dam, the fish find that the impoundment created behind the dam results in habitat alteration, affecting the ability of migratory fish to continue their migration and spawn. Downstream migrating fish are often subject to reduced flows and velocities in the project's impoundment, delaying downstream movement. Discharges from downstream fish passage facilities often concentrate fish, resulting in increased predation. Passage through the components of downstream passage facilities including fish barrier screens, dewatering facilities, transportation flumes, etc. can cause injury to migrating fish. Turbine passage is not benign, entrained fish are subject to injury from mechanical mechanisms, fluid mechanisms (shear-turbulence and cavitation) and pressure changes. Passage through the turbines can also cause disorientation, increasing the fishes' vulnerability to predators.

A fishway provides a solution to the problem of a dam separating a migratory fish from its upstream spawning habitat, and for downstream emigrating fish. As a general policy, natural resource agencies are mandating the installation of both upstream and downstream fish passage at man made obstructions in waters where migratory species are currently or have historically been present; at some locations, downstream fish passage is mandated to allow the movement of "resident" fish species. The mandate for the installation of fish passage at a hydroelectric project may be a condition of license for projects regulated by the FERC; or a requirement aimed at maintaining or enhancing the natural resource at federal projects operated by agencies such as the ACOE, USBR, and TVA. For a limited number of hydroelectric projects that do not fall under the purview of the FERC or a federal agency, the installation of fish passage facilities may be mandated by natural resource agencies in response to an owner's plan, wherein it is proposed to implement significant modifications or repairs at a hydroelectric project.

A good, general reference on the function, design, and operational problems of fishways is the *Design of Fishways and other Fish Facilities* (Clay, 1995).

4.5.2 Problems

The problems associated with fish passage facilities are as numerous as the projects requiring passage, and while upstream and downstream passage facilities have problems specifically related to the direction of passage, there are also a number of problems that are common to either type of facility. The problems that occur at fish passage facilities are the result of the means used for fish passage, the type of structure to be used to pass fish, the flow regime under which the facility will operate, and the target species to be passed.

The problems described in this guideline are by no means exhaustive, as each project has its own site specific problems and possible corrective measures. The problems described are considered common and generic to either the type of fish passage facility or a particular function performed by the facility, and the description of the problem is intended as an overview. Often the "overall solution" used to mitigate a particular fish passage problem (example: transportation, behavioral devices, screens) results in an additional set of problems that are associated with the solution. This subsection of the guideline on problems differs slightly in format from other similar sections in that discussion of problems, causes, and solutions are presented together.

a) Upstream Fish Passage

There are a number of ways to provide upstream passage: fish ladders, mechanical devises (elevators/lifts and locks), natural channels, trap and transport, and fish pumps.

i) Fish Ladder

The actual physical structure which allows fish to ascend upstream by swimming or jumping from pool to pool, or climbing, is the ladder. A ladder tries to present the upstream targeted fish with hydraulic conditions that are compatible with its swimming and behavioral characteristics. Ladders consist of an open channel trough with an internal baffle. Various types of ladders use different baffle arrangements, each ladder type and baffle arrangement having been found suitable for certain types of fish.

Ladders are classified based on hydraulic design and function, and include the pool type (vertical slot, pool and weir, Ice Harbor), and chute type fishways (Denil, Alaska Steep Pass, Eel Pass). The various types of ladders have been successfully used by a wide variety of anadromous and freshwater fish. The orifice-weir (variation of the pool and weir) fishways have been used successfully by anadromous salmonids. The Denil provides the most direct route of ascent, while the vertical slot requires fish to use a "burst-rest" pattern to move between pools. Fish move faster through Denil fishways than through vertical slot or weir fishways. While the pool and weir is the most common ladder currently found and has the longest history of use, the vertical slot design is now the preferred type of fish ladder.

<u>Vertical Slot Ladder</u>

The vertical slot ladder, a pool type fishway, derives its name from the use of baffles having an open slot the full height of the baffle. Currently it is the preferred ladder arrangement. The vertical slot ladder allows fish to choose their preferred level of passage through the slot. The slot width itself is varied depending upon the species being passed, with the slot made wide enough to prevent injury to fish if they fall back during passage. Vertical slot fishways vary in width from 6 feet to more than 16 feet depending upon the size of the anticipated passage rate.

The main advantage of a vertical slot fishway is its ability to operate under a wide range of pond fluctuations and to provide the fish with the ability to choose its own preferred passage depth. The fishway does not require adjustments for normal pond fluctuations, and is self regulating in discharge of flow and establishing water levels in the pools. Although there is some potential for debris clogging of the slots, this generally does not cause the fishway to become ineffective. The vertical slot fishways require slightly more flow than Denil type ladders.

Often a "spoiler" plate is installed near the bottom of the slot for two reasons. Firstly, the plate helps stabilize the flow through the slot; secondly, the plate provides an orifice through which some species prefer to pass.

Pool and Weir Ladders

Pool and weir ladders were the first type of fishways developed, and consist of a series of pools with water flowing over the top of the pool's downstream edge (the weir). These ladders are highly susceptible to pond fluctuations and are typically used for salmonid fish species that have a leaping ability. Although relatively inexpensive to construct, because of the need to control water flows, and the inability to pass a variety and even certain fish species, the pool and weir fishways have fallen out of favor.

The pool and orifice type of ladder (known as Ice Harbor, after the first site to use this type if ladder) is a continuous flow gravity fed fishway. The ladder's concept is similar to the pool and weir, but adds an orifice at the base of the baffles and baffle projections into the flowage. Salmonid species have shown a tendency to prefer passage through orifices, and the weir provides an additional passage route for species which do not typically pass through orifices. The main limitations to this form of ladder system are the potential for the orifices to clog, and the need to have relatively stable pond level conditions. The orifices are typically over 2 feet square with the minimum recommended orifice opening at 18 inches square. However, flow over the weir must be relatively constant for the fishway to not develop unstable hydraulic conditions. Typical flows over the weir have an average depth of 12 inches. Impoundment variations as small as 6 inches may require the fishway water levels to require adjustment. One method of adjustment for pond fluctuations is to use a section of a vertical slot fish ladder upstream of the Ice Harbor ladder to regulate flows.

The physical size of the ladder is also dependant upon the anticipated run size; ladders are typically between 10 and 20 feet wide (or more). While the Ice Harbor fishway will typically be of similar size as the vertical slot fishway, the Ice Harbor requires larger flows through the ladder in order to pass fish. Although the Ice Harbor type fishway has been used successfully at various sites, its need for larger flows, and its susceptibility to impoundment fluctuations, means that it does not offer significant advantages over the vertical slot type of fishway, and currently it is not normally considered for fish passage.

Denil Ladders

Denil ladders, a chute type fishway, were developed to overcome the weir type ladder's limitations with impoundment fluctuation; although capable of operating over a wider range of fluctuation, the range is limited to a few feet. The Denil ladder (named after its inventor) is an artificially roughened channel and has been extensively used throughout the world. Denils are typically four foot wide and consist of a series of baffles placed at an angle to the flow and forcing the water to be turned back upon itself. The Denils' slopes vary depending upon the swimming ability of the targeted fish species. Typical ladders are placed in a 17% slope, with some ladders for weak swimmers being placed on a 13% slope. To date, the longest known Denil is 750 feet and ascends a height of 45 feet. Denil ladders require, and have, limited flow capacity and therefore it is likely that such a ladder would require additional flow at the ladder entrance to attract fish.

Denils are capable of operating with impoundment fluctuations of up to three feet. Should the owner agree to limit pond fluctuations to two feet, a Denil ladder would not require adjusting. Should the pond fluctuate more than three feet, additional baffles can be installed to increase the operational range of the Denil.

The main disadvantages of the Denil ladders are its subjection to clogging by debris, and its limited capacity for fish passage. Denils can be constructed from concrete, wood, or steel, and due to their narrower width and geometry, can be installed at sites with tight, steep terrain.

Fish pass quickly through a Denil ladder and fatigue may become an important consideration. Resting pools are required for fish ascending the ladder since the fishway requires the fish to pass one ladder section at a time in its entirety. Resting pools for the fish are normally provided every six to nine feet of vertical lift.

Alaskan Steep Pass

Alaskan Steep Pass (ASP), a chute type fish way, is a modified Denil type ladder with limited fish passage capacity. The main advantages of the ASP are that it is small, lightweight and can be prefabricated, making installation easier than conventional ladders. The ladder slope can be as much as 33% (with a normal slope of 25%), and the ASP has low flow requirements for passage. ASP have successfully passed weak swimmers like the American Shad although the shad do not enter the ASP well. The main disadvantages of the ASP are its susceptibility to pond fluctuations and its passage capacity. The baffles within an ASP are more complex than a typical Denil ladder and therefore can clog more easily or rapidly.

A typical ASP (22" high by 14" wide) can function with only a 12 inch pond fluctuation. However, there is potential to increase the height of the ASP to allow larger pond fluctuations. In theory, when the water level within the ASP becomes too deep for the floor baffles to be effective, the ASP would react in a similar way to a typical Denil ladder. Unfortunately the concept of higher walled ASP's has not been tested, or tried, at any known installation, although the concept was, or is, being considered at a few sites. The capacity of an ASP has not been established although a site in Rhode Island has passed between 50,000 and 60,000 river herring and appears to have reached its capacity. Slatick (1975) estimated the passage capacity of chinook in an ASP was between 650 and 1140 fish per hour. One method to increase capacity is to install multiple adjacent ladders.

As with a Denil, fish pass quickly through an ASP and fatigue may become an important consideration. Resting pools are required for fish ascending the ladder, since the fishway requires the fish to pass one ladder section at a time in its entirety. Resting pools for the fish are normally provided every six to nine feet of vertical lift. The time to travel an ASP fishway at the Cabinet Gorge dam is estimated as being 1-1/2 to 2 hours.

Climbing Pass Ladders

Climbing passes, a chute type ladder, include the Alaska Steep Pass and the Eel Pass which are used to pass eels. Due to the eels' ability to swim and climb, design of eel passageways have incorporated a wide variety of configurations, including roughened flumes, and pipes full of straw or artificial vegetation mats. Eels can be passed through flumes which have been built of wood and aluminum, and plastic and steel pipes have been used. Eel passes can be set on steep inclines due to the eels' ability to climb slopes of 90%.

The overall problem with ladders is that there is not a single type of ladder that will pass all species of fish, due to differences between species, including their physical size, strength, stamina, and swimming speeds. Due to the turbulence within the ladder; fish are subject to disorientation, de-scaling and injury as they brush against hard surfaces, and exhaustion. Ladders, especially long or tall ladders, tend to delay fishes movement through the various pools: the longer it takes for a fish to reach the exit the greater the chance that the fish may tire, be injured, or return to the bottom of the ladder and obstruct the movement of other incoming fish. Fish are also often disoriented and delayed when exiting a ladder as they adjust to the new environment and flow conditions (quieter flows of the discharge channel and impoundment as compared to the turbulent flows within the ladder). Proper placement of the ladder exit can reduce delays in exiting, and the exit should be located away from spillways or powerhouses where the fish may be swept downstream or entrain them in the powerhouse intakes.

Modifications to an existing ladder to reduce the time required to move up the ladder may be assessed using physical or computational models, and often can be performed in trial and error fashion and/or systematically using scientifically defensible methodologies.

Problems associated with ladder civil works are those related to the use of thin walled concrete subjected to erosion by high velocity flows transporting bedload sediments, and possibly freeze-thaw actions. Other problems include those associated with the use of piping systems, and mechanical gates and valves. and their associated operators and electrical control systems.

ii) Mechanical Devices

Fish locks and elevators are mechanical means of raising fish from the tailwater for discharge into the impoundment, and are collectively referred to as fish lifts.

Most of the fish locks constructed to date are in Europe, although a few have been built in the United States. Fish locks operate in the same fashion as locks for ships. The lock's operation includes first closing the upstream gate and allowing the fish to enter into the lower horizontal chamber from the tailrace through the opened downstream gate. At a predetermined cycle time, or when sufficient fish have entered the lock chamber, the downstream gate is closed, the vertical shaft fills with water, and the upstream gate is opened. The fish then swim up the shaft and into the headpond. The primary disadvantages of fish locks are the high cost of the structure and their much smaller capacity compared to ladders and lifts (Clay, 1995)

Fish elevators are similar to fish locks, but in the former fish are lifted vertically in a mechanically driven hopper. The fish enter the elevator from the tailrace led by the attraction water flowing out of the lower entrance. When sufficient fish have entered the bottom entrance channel a "V" trap crowder closes and travels towards the elevator shaft to force the collected fish above the hopper. After the crowder has moved to the full closed position, the hopper is lifted up the vertical shaft by an electric or hydraulic hoist. When the hopper reaches the full up position, a hopper door opens, and the fish are passed into the exit channel where they swim out into the headpond.

Fish lifts require the use of screens, brails, or gates which move through the water to crowd and isolate the fish into the lock or hopper: devices that are all susceptible to deterioration, fouling, jamming, and other problems associated with machinery operating in water or wet environments. Fish lifts, by their mechanical nature, have higher operating and maintenance costs than fish ladders. While fish lifts can be, and are, designed to operate untended, the environment in which the equipment operates and the debris transported in the flows, results in greater personnel costs for tending and maintaining lifts than for ladders.

iii) Natural Channels

Natural channels are a man-made channel designed to allow fish to pass around a dam. The channels are designed and constructed to replicate conditions encountered naturally in the river or stream, and as a result the natural channel does not restrict in the fish species it passes. Natural channels, if properly designed, will function effectively under a wide range of impoundment and tailwater levels, and if properly located, may also be capable of providing downstream passage.

Natural channels may be entirely man-made, or may be constructed in part, using pool and riffles, to take advantage of the site's topography. As opposed to other means of fish passage, natural channels can be designed to discharge a significant flow though the channel, thereby providing a site with a way to discharge minimum or by-pass flows downstream without the need for a gated discharge structure.

Natural channels are not suited for all dams, especially high structures or structures in gorges, as the channel requires a significant amount of ground to accommodate its gradual slopes (greater than 20H:1V). However, the costs of operating and maintaining a natural channel are significantly less that that of any other means of upstream fish passage. The cost of constructing a natural channel may be the same or more than that of a ladder or lift. In 2005 natural channels have not been commonly used, and as a result there is not a significant amount of data on the design and effectiveness of channels in operation. Prior to embarking on constructing a natural channel, consideration should be given to performing physical model tests to establish the geometry and shape of the channel for the prospective site.

iv) Trap and Transport

Fish ladders and elevators can be readily adapted as part of a trapping and transporting operation. At many facilities the fish, either before or after a sorting section, can be passed directly into the headpond, into holding tanks for future movement, or directly into trucks for immediate transportation. Transporting of fish can eliminate the need to construct additional fishways at other blockages located further upstream on the river.

The primary potential advantages of a facility dedicated exclusively to a trap and transport operation, compared to an elevator with direct headpond passage and a trapping and trucking option, is that the much shorter vertical height and hopper size decreases the initial construction cost. Conversely, a dedicated trapping and trucking facility would potentially result in higher operating and transportation costs than ladders and elevators with direct headpond passage.

Another attractive feature of the dedicated trapping and transporting approach is that the trapping facility does not have to be located close to the dam. If convenient downstream location(s) can be found, where the target fish can be attracted and trapped in sufficient numbers, a trapping and transporting approach may offer significant cost savings. Since dedicated trapping facilities are smaller than ladders or elevators, it may be economical to experiment with a number of trapping sites to determine if a suitable collection efficiency can be obtained. Trapping facilities can sometimes be adapted as temporary and/or movable structures, as an economical approach to experimenting with the effectiveness of different trapping locations.

However, trapping and transporting adult migrants is, at times, a highly controversial method of fish passage. There is concern that the delay and crowding in the traps and tanks can cause increased stress on the fish, elevated susceptibility to disease, and increased mortality.

v) Fish Pumps

Fish pumps are used to move fish in aquaculture settings but only a limited number of dam sites have been developed to move adult fish upstream of projects. The types of pumps used are air-lift, screw impeller, jet, and volute pumping systems. Pumping fish has the potential to lead to injury and de-scaling as a result of crowding in the bypass pipe. Use of fish pumps for fish passage is generally not accepted any more

b) Downstream Fish Passage

Survival of juvenile and adult fishes varies according to the route chosen by the fish on their downstream journey. Downstream passage can be provided in a number of ways: by spillage over the dam; transportation of hatchery raised juveniles; screening, collecting, and diverting around the obstruction; and passage though the turbines. Downstream fish passage is a greater problem at sites where there is a large change in the level of the impoundment, or sites where the majority of the river flow is discharged though hydroelectric turbines.

i) Spilling

The spilling of water over or through a dam is the simplest way of transporting fish downstream, past a hydropower project. Spilling can be cost effective when the downstream migration period is short, when fish migration occurs during high river flows, or where spill flows are needed for other reasons, such as maintaining water quality or providing recreational flows.

Problems associated with the spilling method of downstream fish passage include injury by abrasion or impact; increase of dissolved gases in the water (resulting in gas bubble trauma), pressure-induced injury of fish that are discharged through a low level gate; and disorientation and stunning of juvenile fish resulting in increased predation by birds and predatory fish in the tailwater. Spilling of water may also have significant economic impacts if the flows that are released lead to a loss of generation revenue.

ii) Transportation

Transportation as a means of providing downstream passage of juvenile fish encompasses both collection (trapping) and barging or trucking to the release site. Transporting fish around hydropower facilities is used to reduce the loss of downstream migrants as they emigrate through reservoirs, to avoid the impacts of nitrogen super saturation that occurs when spilling water over a dam, to decrease the possibility of turbine entrainment, and to help avoid predation at entrances and exits to fish passage structures.

iii) Diverting

Diverting fish around dams and powerhouses is a common means of providing downstream fish passage. The diverting structures used to direct fish away from the powerhouse intake may included screens, structural guidance devices, and behavioral devices. Once fish are diverted they are guided to a sluice, trough, or collection chamber that transports the fish around the dam to a location where they can be safely discharged into the tailwater on the downstream side of the barrier.

Diverting structures are susceptible to bio-fouling and floating debris and can require extensive upkeep to keep them operational. Preventing the entrainment of juvenile fish in intakes is extremely difficult as their size often requires small screen spacing and low velocities to prevent fish from being injured or impinged fish on the screens or racks. For projects with shallow intakes whose height extends nearly the depth of the impoundment, or projects where the powerhouse forms a "dead end" in a power canal, excluding fish from the intake is even more difficult.

Diverting structures can also include traveling and inclined screens located in the intake structure. These screens direct fish upwards out of the flow into the turbine, and into a collection chamber that allows for their safe passage downstream of the powerhouse. Inclined screens can also be installed in a penstock, with the fish diverted into a by-pass pipe and discharged downstream of the powerhouse.

iv) Turbine passage

The assumption behind the requirement for a fish bypass facilities is that mortality associated with the bypass will be less than that from passage through the turbines. Turbine passage exposes fish to being struck by the rotating turbine blades which can de-scale or kill them, and rapid pressure changes and hydraulic shear forces that which can also cause injury and/or death. Turbine design and operational factors affect turbine mortality rates. Small diameter high speed turbines increase the change of blade strike; Francis turbines with their greater number of "blades" have higher fish mortalities than Kaplan or Propeller turbines; and sites and turbines under high heads result in a more significant and rapid change in pressures as the fish pass through the runner and into the draft tube and tailwater. Large, low head, low speed, pit bulb, propeller type turbines have low rates of fish mortality.

Progress continues to be made in the development of "fish-friendly" turbines that can be intentionally used to pass fish. Changes to the turbine's design include smoothing of surfaces, reducing the gaps between the runner, or runner blades, and non-moving surfaces, decreasing the rotational speed, reducing the height of the turbine above the tailwater, and increasing the depth of the entrance to the penstock.

c) Common Problems

i) Siting

The siting and design of a successful hydroelectric project involves the integration of both engineering and biological information. The location of a fish passage facility is often controlled by issues such as accessibility for construction, operation, maintenance, and monitoring; and cost of construction. Site conditions such as the height of the barrier to be overcome also impart limitations as to the type of fish passage facility that can be used.

Siting of the entrance to upstream or downstream fish passage facilities is often the single most important component of most fishway designs, and it is usually the most difficult aspect to address. A poorly sited entrance can have significant impact on the effectiveness of the facility to pass fish. Ideally, entrances to the fish passage facility should be located where fish are known to congregate due to the obstruction caused by the dam or powerhouse.

The effectiveness of the facility can also be impacted by the volume and velocity of flow, whether the flow is due to high river conditions, or the normal and routine discharge from a powerhouse or discharge gate. High flows, whether distributed along the length of a spillway, or locally at a powerhouse or discharge gate, can mask the attraction flow at the entrance to a fish passage facility, thereby confusing the fish and delaying their passage though the facility.

Flow discharged from a powerhouse or dam both attracts and delays the passage of fish. At downstream fish passage facilities, the fish are attracted to the velocity of the flow towards and into the intake structure, and discharge over or through the dam. At upstream fish passage facilities fish are attracted to the flows being discharged from the turbines and spillway-sluice gates, conditions which, if occurring across a wide distance, can make it difficult to attract fish to the entrance of the fish passage facility.

ii) Design

The design of an effective and functional fish passage facilities includes integration of both engineering and biological design. Engineering design takes into account the site's: flood protection (impoundment and tailwater); bed (sediment) and ice load; and type and quantity of floating debris. Accessibility of the facility both during construction and for operation and maintenance are also key factors in the layout of the fishway, as are impacts from operation and maintenance of the hydroelectric project. The effectiveness of the fishway's operational function or reliability is controlled by attention to the selection of the materials used in its construction, and attention to detail in the design of the fishway's components and systems.

Biological design addresses the requirements of the fish species that are expected to use the fish passage facility, it identifies the fishes': life cycle and time when they will move thought the fish passage facility; swimming speeds; behavioral patterns including preference for water temperature, velocities, and swimming depths. Biological design also includes the site's water temperature, D.O., siltation and/or turbidity, pollutants, and the determination of the upper and lower limits of flow at which fish passage will be operational.

Criteria have been established for use in "sizing" or "designing" a fish passage facility (Bates 1992, Bell 1984 and 1991, Clay 1995). Sizing and siting criteria are usually mandated by a governing natural resource agencies, while standards for use in the design of the civil works, i.e. the concrete and steel, are those published by the AISC, ACI, and the USACE. Subsection 4.5.6 identifies references that contain criteria established by the natural resource agencies, and general information and reports on fish passage.

This subsection on Design identifies critical areas in the design of a fish passage facility that impact the function and effectiveness of a fishway including: entrances and attraction flows; biological information and criteria; impingement and entrainment; modeling; and operations and maintenance.

<u>Entrances, Exits, and Attraction Flows</u>

Not only is the location of the entrance to a fishway critical, but so is the design of the conditions at the entrance. The entrance must be designed to attract fish in a timely manner, and the adequacy of attraction flow is the most important element. Fish are attracted by velocity of the discharge from the entrance.

If there are no other overriding stimuli, fishway design assumes that the physiological processes that guide migrants back to their natal waters are a combination of odor response and the behavioral response of swimming upstream against flow (Clay, 1995). Factors that determine entrance effectiveness include; jet momentum of the discharge, its shape, and alignment (Bates, 1992 and Clay 1995). To make an entrance flow noticeable to fish at the greatest possible distance, the flow needs to be discharged at the highest velocity that does not prevent or discourage entry of fish. Bates (1992) indicated that the greater the momentum of the jet or the distance that water flows out of a fishway the more successful it is likely to be at guiding fish to the entrance. Depending on the site and other factors, "attraction flows in excess of that required for the fish to move through the fishway may be added at the entrance to increase velocities within the entrance channel or strengthen the entrance jet. The entrance flow must be sufficient to compete with spillway or powerhouse flows for fish attraction.

Although there are no specific entrance flow criteria, there are some general guidelines that are usually employed. Generally, the accepted minimum standard entrance velocity for salmon is 4 fps (Clay 1995). In turbulent locations, particularly those below powerhouses and spillways, velocities up to 8 fps may be used to enhance collection effectiveness. For American shad and river herring velocities of 4 to 6 fps are routinely used at the entrances of east coast fishways (USFWS 1994).

As long as there is flow through a fishway almost any velocity can be obtained at the entrance by adjusting its size larger or smaller. However, if the entrance is made too small, fish may not be able to locate it, particularly in locations where flow near the entrance is turbulent. Fishway entrances can either be a full or narrow weir, an orifice, or a vertical slot. The size of the entrance is normally dictated by natural resource agencies who determine the size of the entrance based on the size and species of fish to be passed. For fishway installations in turbulent locations, the entrances are located in a minimum water depth of 4 ft combined.

At upstream fish passage facilities, the attraction flow is released in the ladder's entrance chamber, located at the downstream end of the ladder in the tailwater of the dam or powerhouse. Transportation flows may be in the order of 25 to 100 cfs, depending on the fish species and size of the fish run, and the added attraction flows may be an additional 100 cfs to several thousand cubic feet a second of discharge. In addition, the hydraulic energy in the added attraction flow needs to be dissipated prior to being discharged into the ladder's entrance chamber.

At downstream fish passage facilities attraction flows are needed at the entrance to the facility which is located in the impoundment. The USFWS normally requires that the volume of the attraction flow is equal to 2% of the hydraulic capacity of the turbine discharge, inclusive of a flow of 25 cfs to 50 cfs to transport fish around the barrier. Screens and guidance devices may be used to guide and direct fish to the entrance structure, and overlays may be installed on the intake trash racks to prevent fish from being entrained.

The exit from an upstream fishway requires proper placement to reduce delays in exiting the fishway into the impoundment, and it should be located away from spillways or powerhouses, where the fish may be swept downstream or entrained in the intakes. The exists for downstream fishways are locate in the tailwater of the dam or powerhouse. The exit should be sited to reduce predation of the exiting fish which may be disoriented from passage through the facility. Downstream fishways often utilize a pipe to by-pass the dam or powerhouse. When returning fish to the tailwater, they should be discharged in a manner that does not allow them to become separated from the water column to free fall and "belly flop" on the surface of the water.

Biological Information and Criteria

The fish species to be bypassed by a hydroelectric project are determined by natural resource agencies based on their restoration and management plans. The plans list target species and ages, prioritize passage of the target species, and include design populations for those species. The plans also identify the type and an approximation of other migrants that are anticipated to use the fishway, and the plan typically provide information on the timing and number of fish moving through the fishway for seasonal and daily periods of migration.

Depending on the project and agencies' fish passage goals, the swimming performance data of the weakest swimming species targeted for passage is often utilized in the design of the fishways. Behavioral information including; migratory patterns daily and/or seasonal, schooling and migration information, migratory clues (i.e., water temperature and flows), and avoidance behaviors are also incorporated when designing a fishway. Factors that affect swimming performance must also be considered, particularly those factors that have been shown to affect passage inside a passageway; they can include: water temperature, dissolved oxygen, hydraulics and turbulence, source water migratory cues, ambient light and/or shadows, sudden noises, and pollutants.

Fish species do not move through the water column at the same depths or at the same speeds. Bottom swimming species such as eels and sturgeon are attracted to entrances and flows near the streambed. Top swimming species such as alewife, shad, salmon, and herring are attracted to entrances located in the top third to quarter of the water column. It is difficult, and depending on the site, it may be impossible, to design a fish passage facility which enables the passage of both top and bottom swimmers in a single facility.

Ladders used for upstream fish passage, in addition to not being able to pass all surface swimming fish equally, may not be able to pass bottom swimming species at the site. Fish lifts are indiscriminate and the most effective in the passage of top or bottom swimmers, as long as the attraction flows are located to attract those species of fish.

Impingement and Entrainment

Impingement is the act of a fish becoming stuck on a screen or intake trashrack. Fish are attracted to the velocity of flow towards these barriers, and if the velocities are high, in excess of 2.0 fps, the fish may not have sufficient strength and speed to swim away, or they may become exhausted in trying to prevent themselves from being drawn into the intake.

Entrainment is the act of a fish swimming or being drawn into an intake. Intakes with screens or trash racks with a clear spacing of three or more inches allow fish to pass through, and subsequently become entrained in an increasing velocity as the flow moves into a penstock or turbine.

Modeling

The hydraulics (flow) of a site normally defines the scale of the fishway and the location of its entrance(s) and exit(s), site hydraulics also to determine fishway flows. To understand how flows will affect the operation of the fishway, the site's hydraulics should be observed and/or modeled over the entire range of flows that the fishway is intended to operate.

Modeling can be performed using numerical or physical models, depending on the type of information needed. Hydraulic modeling of a fish passage facility can be performed to assess flow patterns in and from upstream and downstream fishways. For example, modeling may be performed to examine flow patterns that result from the use of screens, or other structural guidance devices; the layout and size of baffles within a fish ladder; the velocity at the entrance to a fishway; or the geometry and shape of a natural channel. Modeling is not routinely performed as part of the design of a fishway, but rather to address, or evaluate, specific or anticipated problems with a fish passage facility

Operations

In order to reduce operating costs, as many functions of a fish passage facility as possible should be automated. Some of the functions that can be automated include:

- opening or closing of a gate or valve to maintain a given flow or water level.
- raising and lowering of a fish lift on a timed cycle, including the associated operation of crowder screens and gates.
- flushing and cleaning of trash racks and screens.
- counting of fish using acoustical or video methods, with video also providing the means to identify the type of fish passing though the facility.
- directing or shunting fish to a holding location for subsequent biological study.

A facility can also be designed and constructed with the means to view and observe fish as they move through the fishway. Provisions can also include means to selectively isolate fish for subsequent removal for tagging, scientific study, and transportation upriver and or to a hatchery.

Maintenance

Fish passage facilities are an assembly of electrical and mechanical components: gates, valves, penstocks and piping, concrete structures, trash racks, operators, control systems. All are features with functions that occur elsewhere in a hydroelectric project, and with the same requirements for maintenance. Consistent performance of any well-designed fishway is largely based on maintenance and regular observation of operation. The use of erosion and corrosion resistant materials can result in significant reduction in operational costs and extend the life of the facility. Fish passage facilities, especially fish ladders, may experience problems associated with the use of thin walled concrete which are subject to erosion by high velocity flows transporting sediments and bed load materials, and freeze-thaw actions.

Debris is possibly the single greatest problem, other than those related to mechanical systems operating in water, that is likely to occur during operations at both upstream and downstream fish passage facilities. Accumulation of debris can result in increased loads on structures; cause blockage and obstructions within the fishway; alter flow conditions and velocity; slow or prohibit fish movement though the passage; and jam and prevent the operation of mechanical components.

While fish passage facilities are equipped with trash racks or grizzly racks, the spacing between the bars is often ten or twelve inches clear; too large to exclude leafy and small woody debris and much of the manmade material found on waterways. Because of the small size of the openings and passages in a fishway, debris removal from a fishway often requires greater level of manual effort than does removal of debris at the hydroelectric plant. An ideally sited and designed fish passage facility would not only include trash and log booms, and trash rakes, but would also include the means to flush and sluice debris from the entrances and intakes.

iii) Overlays, Screens, Structural Guidance Devices, Behavioral Devices

Overlays, screens, structural guidance devices, and behavioral devices may be used at both upstream and downstream fish passage facilities.

Overlays

Overlays are added to screens or trashracks at an intake structure to reduce the clear width of the openings. To exclude juvenile fish, natural resource agencies typically prefer a maximum clear opening of 1 inch. Overlays can be a permanent addition to an intake or may be removable and installed only when fish are migrating downstream. Because overlays result in a reduction of the clear opening of the intake, the velocity at the face of the racks will be increased.

Overlays may result in increased headlosses at the trashracks, due to the reduction of the clear opening, and hence an associated loss of power generated. Smaller openings in the trashracks will also result in the increased accumulation of debris, and increased costs to clean the trashracks.

Screens

Screens are physical barriers generally used in conjunction with a bypass to facilitate passage. Screens are made of various materials, based on the application and type of screen, which may include perforated plate, metal bars, wedgewire, or plastic mesh. Screens are designed to slow velocities and are positioned to guide fish to a bypass. There are many types of screens, which include submerged traveling screens (STS), vertical traveling screens, simple inclined screens, rotating drum, Eicher screen, modular inclined screen (MIS), fixed, and barrier nets.

Screens may be installed on a freestanding structure independent of the powerhouse or dam. The structure may be located across a canal, intake, or tailrace. Screens require frequent maintenance to maintain integrity (such as with barrier nets) and keep them clear of debris which impedes flow, or if allowed to accumulate excessively, can overstress and fail the supporting structure. Screens with moving components also have problems common to equipment operating in water or wet environments.

Structural Guidance Devices

These guidance devices do not physically exclude fish from intakes, but instead create hydraulic conditions in front of the structure. Fish respond by moving along the hydraulic conditions to the bypass system. These devices include the angled bar trash rack, louver system or array, and surface collectors.

Structural guidance devices may be installed on a freestanding structure independent of the powerhouse or dam. Problems with structural guidance devices are the same as those occurring with screen overlays.

Behavioral Devices

Behavior-based devices are touted as being less expensive than physical screening devices and easier to install than conventional screening devices. Another presumed benefit is that these technologies can be used with little disturbance to the physical plant, or project operation. These guidance technologies employ sensory stimuli to elicit behaviors that reportedly will result in downstream migrating fish avoiding, or moving away from, areas that can reduce fish survival. These devices include acoustics, strobe and mercury lights, electric fields, and bubble curtains.

Behavioral devices are not equally effective on all species of fish; some species will react to one type of device, or a specific frequency or intensity of field, and not react to others. Behavioral devices are mechanical and electrical systems that are required to operate underwater, and as a result maintenance of the systems can be difficult and expensive.

iv) Life Expectancy

Fish passage facilities are constructed using steel, concrete, and other materials, and they contain a significant number of mechanical and electrical systems. The life expectancy of the materials and components that make up a fishway is shorter than that for similar components that can be found elsewhere at a hydroelectric project; primarily because of the extreme conditions (water flow and climate) to which the fishways are subjected, the "lightness" of the materials used, the frequency of operation, and the difficulties in performing maintenance due to its design and layout.

The life expectancy of a fishway may be a pre-defined expectation or requirement, but more often its life is inadvertently established at the design stage by the level of attention given to selection of materials used in the construction, and attention to the design of its components and operation. Ultimately, the actual life of the fishway is controlled by the frequency of proper maintenance. Fish ladders that have limited mechanical systems have been constructed since the early 1900s, and there are a number of ladders constructed in the early 1960s that continue their function with little maintenance; although in general, fishways have a higher maintenance and operational cost than do other major features of a hydroelectric project.

4.5.3 Corrective Measures

The corrective measures associated with fish passage facilities are, in general, described in Section 4.5.2 – Problems. Because fish passage facilities are a compilation of structural, mechanical, and electrical components (i.e. concrete and steel; gates, valves, penstocks, and trash racks; operators and control systems) and functions that are also used elsewhere in a hydroelectric project, corrective measures and solutions to problems associated with them are described elsewhere in the guidelines.

4.5.4 Opportunities

Opportunities for improving the operation, or operational efficiency, of fish passage facilities, or extending their life, are as numerous as the number of facilities in operation, and the number of individual components utilized in the operation of a particular facility. Frequently the automation of an operational function, the change of a single component (pump, control, gate or valve), or the use of erosion and corrosion resistant materials may result in a significant reduction in operational costs and extend the life of the facility. Because fish passage facilities are an assembly of electrical and mechanical components (gates, valves, penstocks, concrete structures, trash racks, operators, control systems) and functions that are used elsewhere in a hydroelectric project, opportunities for improvement of those individual components and functions are described elsewhere in the guidelines.

This section on opportunities focuses on means that have been used to recover energy that would otherwise be lost in the operation of fish passage facilities. At hydroelectric plants discharging flow from a dam that does not generate power equates to lost energy and revenue. Fish passage facilities, be they upstream or downstream, require the discharge of flows to transport and attract fish; therefore opportunities exist for the recovery of a portion of the lost energy, particularly from the discharge of "added attraction flows". Added attraction flows are those flows discharged to ensure flow and velocity of discharge from the entrance is sufficient to compete with spillway or powerhouse flows for fish attraction; added attraction flows are in addition to the flow needed to transport the fish through the facility. The added attraction flow is often discharged with an energy head nearly equal to that on the hydroelectric turbines and the energy in that discharge can be partially recovered, as the added attraction flow is devoid of fish.

Downstream Fish Passage Facilities

At downstream fish passage facilities the attraction flows are needed at the entrance to the facility that is located in the impoundment, not in the tailrace. To reduce or recover energy from the discharge of attraction flows at a downstream fish passage facility, the volume of the attraction flow that is discharged and not available for power generation needs to be reduced while still maintaining the required entrance velocity. One means of recovering the energy is to eliminate the discharging of the attraction flow by continually recirculating the flow between the entrance to the fish passage facility and the impoundment. The required volume of attraction flow can be provided in this way by using low head, high volume mixer pumps. These pumps would be located adjacent to and immediately downstream of the entrance. The pumps would be separated from the fish passage way by screens with sufficient area to prevent impingement of the fish. While the cost of the energy recovery system, and the costs to operate the pumps, is not insignificant the amortized costs may be less than the revenues that would be lost by discharging the added attraction flows.

A second opportunity for energy recovery at downstream fish passage facilities involves the installation of a small turbine on the downstream side of the fish passage facility, at tailwater level. However, the energy recovered by that equipment is done so at a lower overall generating efficiency than that for the primary power generating turbines. In addition, the volume associated with the attraction flow is low, making for limited options in the selection of a power turbine. The intake for the turbine would be located in the impoundment, adjacent to the entrance of the facility. The intake would be screened and would require sufficient area to prevent impingement of the fish on the screens.

Advancements are being made in the development of fish-friendly turbines, designed to pass fish through the power flow without significant levels of injury or mortality. Research into the development of environmentally advanced hydro turbine design concepts or "fish-friendly turbine" began in mid-1990's under the direction of the U.S. Department of Energy (Franke et al 1997). In 2005 the development of the fish-friendly turbine was just entering full scale prototype phase; and the types and size of fish that could be passed, and the heads, flows, and efficiencies over which the turbines would operate have not yet been demonstrated under actual operating conditions. Even if it is demonstrated that fish-friendly turbine can be developed to meet power generating needs, they will only present the opportunity to generate power and pass fish for downstream, outward migrants.

Upstream Fish Passage Facilities

The added attraction flow used in upstream fish passage facilities is released in the ladder's entrance chamber which is located in the tailwater at the downstream end of the ladder. As a result of the hydraulic energy in the added attraction flow, the energy needs to be dissipated in a diffuser chamber prior to being discharged into the ladder's entrance chamber.

Because the attraction flows is discharged into the tailwater at the base of the dam, the head differential between the impoundment and the tailwater may be sufficient to allow the installation of a small hydroelectric turbine to recover energy that would otherwise be lost. The added attraction flow would be piped to the turbine and subsequently discharged into a diffuser chamber that is located inside the entrance of the ladder. While the turbine reduces the energy in the attraction flow by converting it into electrical power, the fish passage facility still needs to have the means to allow the discharge of the attraction flows even if the turbine-generator equipment is not operational. Such a turbine installation will increase the construction and operating costs of the fish passage facility, and depending on the value of the energy being lost in discharging the added attraction flows, the installation of a hydroelectric turbine may not be economical.

A second opportunity for energy recovery is to use recirculating pumps to provide the added attraction flow required at the entrance to the fish ladder. Energy lost is reduced by "continually recirculating the attraction flow" between the entrance of the fish passage facility and the tailwater. The required volume for the added attraction flow can be provided by recirculation using low head, high volume pumps. In contrast to the situation with downstream facilities with recirculating pumps in the impoundment, the pumps for the upstream fish passage facility are located in the tailrace adjacent to and discharging into the ladder's entrance diffusion chamber upstream of the entrance gate. The size of the civil works for such a system can be significant, and while the cost of the energy recovery system and the costs to operate the pumps is not insignificant, the amortized costs, once again, may be less than the revenues that would be lost by discharging the attraction flows.

4.5.5 Case Histories

No. 1 Energy Loss to Fish Attraction Flows
McNary Dam (MWH, 1996)

The United State Army Corps of Engineer's McNary Dam, located on the Columbia River, USA, was largely completed in 1953 when the first generating unit was placed into service. In addition to the dam, powerhouse, and spillway structure, there is a navigation lock and two fish passing facilities, one on the northern (Washington) shore, the other on the southern (Oregon) shore.

Construction of the north shore fishway facilities was completed and operation commenced, in 1953. The original installation consisted of a conventional fish ladder, a vertical fish lock, a holding pond, and the Attraction Water Supply System (AWSS). The fish lock was constructed so that fish could actually be "lifted" up from the tailwater to the reservoir, if the fish were reluctant to use the fishway. The fish lock was considered an experimental facility, and after several years it was abandoned, due to the complete success of the fish ladder, coupled with the cumbersome mechanical operation associated with using the fish lock.

The fish ladder receives a flow of approximately 180 cfs from the reservoir through its top portion. This flow is augmented in the lower portion of the fish ladder by an average AWSS flow of approximately 1,748 cfs. This water enters the fish ladder through concrete culverts and fourteen supply ports (diffusers) spaced along the lower portion of the fish ladder. The average AWSS flows represent less than 2 percent of the average Columbia River flow at the McNary Dam.

The performance of the present AWSS has been fully satisfactory for upstream migrating fish passage. At the time the McNary Dam project was designed and built, much less consideration was given to the downstream passage of anadromous migrants through the system. The smolts migrating downstream through the current high-pressure system experience an environment that is extremely hazardous to their survival. The mortality rates for fish in the original system exceeded 90 percent. Modification of the existing AWSS, the fish lock bypass, and turbine route reduced smolt mortality from an estimated 90 percent to between 6 to 14 percent.

In September 1991 the Northern Wasco County People's Utility District obtained a FERC license to develop a hydroelectric facility, which utilizes the energy wasted from the AWSS. The McNary Dam Washington Shore Fishway Hydroelectric Project is located between the navigation lock and the existing spillway, on the AWSS to the McNary Dam Washington Shore Fishway. The existing AWSS was designed to reduce the head available at the fish ladder supply ports from the 60 to 70 feet available, to the 0.3 feet needed. The new turbine uses the available head to produce energy.

Due to the locations of the existing AWSS water supply conduits, non-overflow gravity dam, fish lock, and fish ladder, the space available for the new powerhouse and turbine water passages was very limited. Several arrangements were evaluated before the final arrangement was selected. Due to the small powerhouse size, it was challenging to find sufficient space for all of the required powerhouse equipment (Figure 4.5-1).

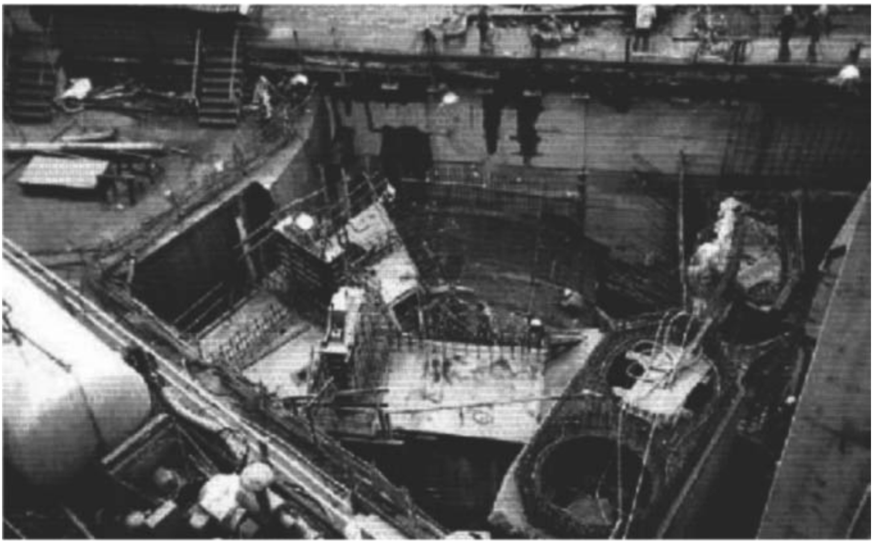


Figure 4.5-1 McNary Attraction Water Turbines (courtesy of MWH)

A typical vertical shaft turbine, with an elbow type draft tube arrangement, has the draft tube exit located approximately 180 degrees from the semi-spiral case inlet. Such an arrangement would have meant expensive tunneling and shoring work under the fish ladder for installation of the draft tube. It would also have required supply tunnels to direct the turbine discharge from the downstream side of the fish ladder, back to the fish ladder water supply ports on the upstream side. For these reasons, it was decided to rotate the draft tube so that it is directly below the intake conduit, thus avoiding the tunneling work underneath the fish ladder. To fit the draft tube into the triangular shaped space between the fish ladder and the fish lock structure, the powerhouse was sited as far as possible to the north.

The unit setting was fixed approximately 15 feet lower than would normally be selected, allowing the draft tube to sit below the foundation level of the fish lock. It also allowed sufficient vertical space between the intake conduit and the draft tube for the draft tube bulkhead. In addition, this deeper setting will prevent cavitation of the runner, and therefore increase smolt survival. The rotating speed was selected to be 211.8 rpm, which is several steps slower than would normally be selected, resulting in a slightly larger unit. The slower speed was selected to reduce the water velocities through the turbine, pressure gradients in the unit and pressure pulsations in the draft tube, and to lessen the possibility of vibration, which could be considered harmful to the fish.

When the turbine is not operating, the AWSS water will automatically be diverted through a bypass system composed of supply conduits and the modified fish lock. The bypass water is conducted into the fish lock structure through two concrete culverts and vertical steel pipes, which turn into the fish lock as horizontal pipes. The water flows into the fish lock from these horizontal pipes, which are open at the top, allowing water to upwell out of the pipes and spill over four weirs. The bypass water flows down the fish lock, which is equipped with four banks of beams (energy reducing shelves) to dissipate the head. The water leaves the fish lock through new orifices in the fish lock walls and enters the same fish ladder supply pool supplied by the turbine.

Several provisions to mitigate injury to fish were included in the powerhouse design. Materials, paints, etc. which have no known harmful effects to fish were selected for the powerhouse, and associated equipment that will be in contact with river water. Greaseless (self-lubricated) bushings are used for the equipment to facilitate maintenance, and to preclude contamination of the water with grease. The powerhouse is provided with oil separation equipment to preclude contamination of the river water with oil, and oil containment provisions are provided at oil storage locations, and around the main power transformer.

The McNary Fishway Hydroelectric Project, completed under a design-build contract in 1996, is the result of many years of planning, work and cooperation between many parties. It better utilizes an existing resource, with the benefits of power generation and an improved system for accommodating the migration of the anadromous fish. The Project long-term benefit is the reduction of juvenile fish mortality by 80 to 90 percent.

No. 2 Entrainment of Juvenile Fish
Puntledge Project, (BC Hydro, 1995)

In 1913, the Puntledge Hydroelectric Project was constructed on the Puntledge River, on Vancouver Island, British Columbia, Canada. The project, constructed to supply power to local coal mines, consisted of an impoundment dam, a low diversion dam and intake structure, and a flume and penstock system to convey water to a powerhouse 6.8 miles downstream of the dam. In 1956, the project was expanded, a 24 MW powerhouse was constructed, and the flume and penstock system was replaced with a 3.2 mile long, 12 foot diameter, wood-stave penstock.

The original project and subsequent redevelopment had a significant impact on salmon stocks. The creation of the impoundment, Comox Lake, affected the production of coho and Chinook salmon, and steelhead trout, which spawned in the tributaries to the impoundment; considered to be one of the more important salmon production areas of the east coast of Vancouver Island. The expanded project reduced the flow in the river by 300 cfs. While provision for upstream passage was made at each dam, downstream passage through the turbines resulted in unacceptably high (60%) juvenile mortality which needed to be resolved.

Records from the 1950's showed several alternative layouts for fish screens, and, in anticipation of fish screen construction, the power intake section of the dam had been constructed with four intakes. Two of the four intakes were to be used for power generation, converging immediately downstream of the dam into the penstock. The other two intakes were capped on the downstream side of the dam.

Previous attempts to divert juveniles away from the penstock intake had been either unsuccessful, or only a short-term solution. In 1992 an initial study of means to divert juveniles away from the penstock considered the use of louvers, drum screens, vertical screens, and the Eicher screen. Conceptual layouts and cost estimates of each of the screens showed the Eicher screen to be the least expensive option at about $4 million. The vertical screen was estimated to cost in excess of $8 million, while the drum screen ran into physical site constraints, resulting in a higher cost than that of the vertical screen. The alternative of not generating during the migration was also considered, and discounted, as it had an estimated present value approximately six times the cost of screening.

Based on the cost estimates it was concluded that a penstock screen, of the type patented by George Eicher, would provide the most economical solution to construct downstream fish passage at the Puntledge Diversion Dam. A critical factor in the choice of screen location was the length of time the generating plant would be out of service for the screen installation.

The Eicher screen consists of a wedge-wire screen installed in a steel penstock at a shallow angle to the flow, and passes fish through a bypass pipe branching from the top of the penstock. The design is unique in two ways: in that it uses much higher approach velocities at the screen than other conventional screens, and the travel time along the screen is very short (Figure 4.5-2). In 1992 the Eicher screen was a fairly new concept in screening, with only two precedents in existence: the Sullivan plant at Willamette Falls near Portland, Oregon in 1982, and the Elwha project near Port Angeles, Washington, in 1989.

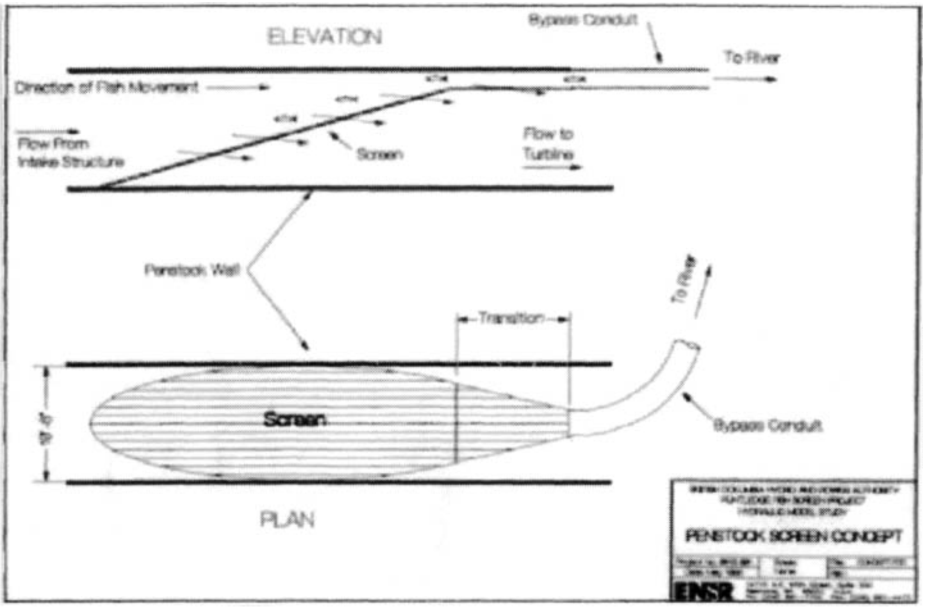


Figure 4.5-2 Penstock Screen – Puntledge Project (courtesy of BC Hydro)

Consideration was given to installing a single screen in the existing 12 foot diameter penstock. To meet the approach velocity criteria, the diameter would have to have been increased to 14.33 feet, which would have been significantly larger than the 9 feet diameter prototype at Elwha. The low gradient of the penstock downstream of the dam did not allow for such an expansion of the penstock for a considerable distance, and there was concern that the screen might not function as well for fish that would have to travel a considerable distance in a totally dark pipe. As the Elwha screen was very close to the intake, it was decided that the location at Puntledge should be as close to the intake as possible, using two 10.5 feet diameter penstocks.

At the start of final design it was judged that, to obtain a uniform velocity distribution across the screen, there should be no horizontal bends upstream of the screen, and the screen should be located a minimum of ten pipe diameters downstream of the intake structure. These criteria would not allow the screens to fit within the existing penstock trench and would have required a relocation of the training wall which separates the penstock trench from the river. A location three pipe diameters downstream of the intakes would allow construction to be contained within the existing penstock trench.

To resolve the location dilemma, a 1:10.96 scale hydraulic model of the forebay, intake structure, penstocks, screens and bypasses was constructed. The model was designed to be operated using Froude scaled velocities as well as full scale velocities. The flow patterns in the forebay, intakes and penstocks, up to the screen location, were studied while operating the model according to Froude similitude laws. Through-screen velocity measurements and head loss measurements were collected while operating the model at full scale approach velocities. Tests were conducted to ensure that the flow patterns immediately upstream of the screen in the full scale velocity tests, were similar to those in the Froude scale test.

The hydraulic model indicated that the improvements gained by locating the screen ten pipe diameters downstream were small when compared to the additional construction cost. A screen location of three diameters downstream of the intakes was therefore chosen.

A second task for the hydraulic model study was to determine the benefits of varying the porosity of the screen. There were indications from the work at Elwha that gradually reducing the porosity of the screen as the bypass entrance was approached might help achieve the 3:1 sweeping to normal velocity criteria. Although some small benefits were demonstrated, they were not considered sufficient enough to offset the more complicated fabrication and additional headloss. A standard Johnson stainless steel wedgewire screen with a uniform porosity of 58% was chosen.

The limited available head of 1.5 feet posed a problem for maintaining pressures above atmospheric in the bypass pipes. The bypass entrance, therefore, was set within the crown of the penstock and the penstock was tapered until the rectangular bypass pipe had fully exited the crown of the penstock. The invert of the penstock was kept constant throughout this transition. Each bypass discharges 25 cfs. The screens are cleaned by rotating them about a horizontal trunnion at their mid point. The backflushing action on the wedgewire provides an effective method of cleaning. Rotation is accomplished by two hydraulic cylinders attached to the screen frame each side of the bypass entrance. An adjustable timer controls screen rotation.

The project also included the design and installation of an evaluation facility for collection and study of the fish being bypassed. The facility, which can be connected to the bypass pipe from either screen, consists of a series of energy dissipation tank, wolf traps, collection tanks, holding tanks, and an evaluation tank. Fish are returned to the river via an outfall pipe.

Two extensive fish mortality studies were done in 1993 and 1994 to test the effectiveness of each of the two Eicher Fish Screens that were installed. The primary objectives were to determine rates of direct and latent mortality to wild and hatchery juvenile salmon, and other populations, moving past the screen. The secondary objectives were to determine the sub-samples of wild population (in order to characterize the degree of scale loss associated with passage along the screen), and to determine the timing and estimate the numbers of wild juvenile salmon in the 1993 run. “Direct mortality” is defined as the number of recently dead fish recovered primarily in the collection tanks of the evaluation facility. “Latent mortality” rates refer to fish that die within 96 hours after collection. Mortality as a result of backwash was also observed. As a result of the second study, a variety of recommendations were made for the operation of the Eicher screen, including decreasing fish impingement on screens by cleaning the trash rack at the intake system more often.

The study indicated that the bypass system was effective at increasing the survival of juveniles at the Puntledge Project, with a total mortality rate of less than 1% and a (turbine) bypass rate of 99.8%. Reasons cited for the project’s success were: establishing the design criteria via a workshop with the fishery agencies and operators of the Elwha facility; the use of a hydraulic model to optimize the overall layout, location and design details to confirm the match between hydraulic performance of the final design and project criteria; and minimizing plant outage and loss of generating revenue by fully assembling and testing the screen sections in the shop prior to shipping to the site. (The outage was 2.5 weeks)

No. 3 Outdated Fish Bypass System and Evaluation Facility Bonneville Project (USACE, 2000)

The Bonneville Project is located on the Columbia River, 40 miles east of Portland, Oregon. The project consists of a dam, two powerhouses, a navigation lock, and recreational and visitor facilities. The dam is a concrete gravity overflow type with a spillway of 1450 feet and 18 tainter gates. The first powerhouse and dam were completed 1938, and the second power house was built in 1982. The two Bonneville powerhouses generate about 5 billion kWh of electricity each year. The project has fish ladders for both powerhouses.

Downstream fish passage systems at the Bonneville Second Powerhouse (B2) did not meet modern criteria. Juvenile migrants can pass Bonneville Dam by three principal paths: through the turbines at each powerhouse, over the spillway, and through the downstream migrant facility at each powerhouse. Juvenile salmonids and other fish migrating downstream are partially protected from the turbine intakes. Submerged traveling screens and vertical barrier screens, upstream of the turbines in the dam’s forebay, intercept fish and guide them into a bypass system of flumes and pipes where fish may be taken for sampling before being discharged downstream of the dam tailrace.

Based on current criteria, sampling facilities within the existing bypass system were deficient from both biological and technical standpoints. These deficiencies included injury to fish from the sampling systems, induced stress to fish from mechanical components of the systems, non-representative sampling, poor system reliability, and inability to monitor all migrants using the bypass. The fish transportation and screening systems were also deficient, because channel velocities were out of criteria, water supply was insufficient and there were high screening velocities, high outfall predation and mortality, and system delays.

Systems were modified to meet current standards through $60 million dollars of improvements. Within the powerhouse, improvements were made to the diversion and collection facilities. The changes included upgraded channel orifices, a new inclined dewatering screen, a retrofitted add-in water system, modified channel geometry, and upgraded emergency relief gating.

The new Monitoring/Sampling Facility included primary dewatering screens with a large-fish-and-debris separator, a control weir structure, sample flumes, Passive Integrated Transponder tag equipment, trim screens, holding tanks, fish lifts, preanesthetic systems, anesthetic systems, anesthetic treatment systems, recovery tanks and a head box.

To move juvenile fish from the powerhouse to the outfall two miles downstream of B2 a transportation flume was constructed (Figure 4.5-3). The design featured a flume layout that utilizes below-grade construction to the maximum extent possible. The transportation flume was arranged to incorporate a juvenile monitoring facility in the vicinity of the outfalls. The outfall features two release points: one for high tailwater elevations, and one for low tailwater elevations.

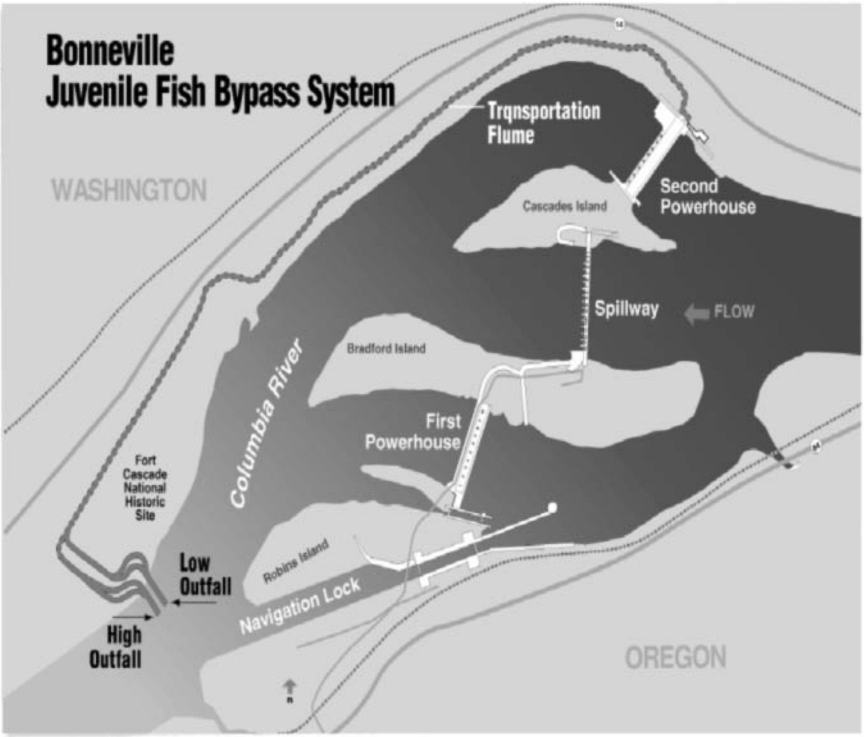


Figure 4.5-3 Juvenile Fish Bypass System (courtesy of Portland District, USACE)

No. 4 Leaping Fish
John Day Lock and Dam (USACE, 2000)

John Day Dam is located 216 miles upstream from the mouth of the Columbia River near Rufus, Oregon. The project consists of a 675-foot long by 86-foot wide navigation lock, 1252-foot long spillway, 1975-foot long powerhouse, and fish passage facilities on both shores. Various recreational facilities are provided along the shores of Lake Umatilla and on the John Day River. Lake Umatilla, impounded by the dam, extends upstream about 76 miles to the foot of McNary Dam.

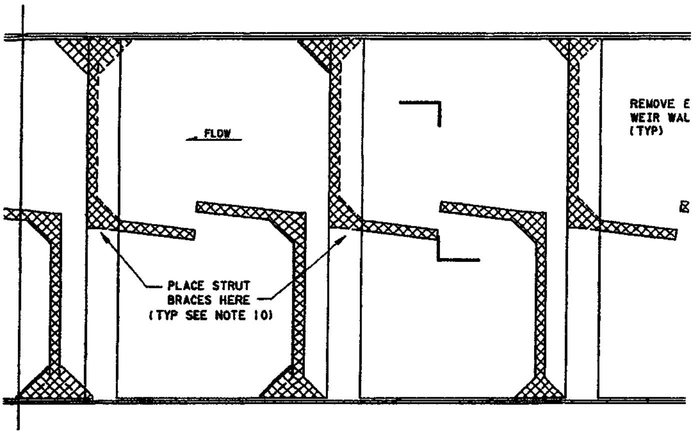
A significant number of the adult salmon and steelhead migrating through the fish ladders at John Day Dam exhibit jumping behavior not common at other fish ladders on the main stem Columbia and Snake Rivers. Specifically, the fish tend to jump out of the water in the first few pools of the control section of the South Shore Ladder, and, as a result of this, many fish are injured, or killed, either from striking structures as they fall back into the ladder, or from landing outside the ladder structure and becoming stranded. Since this behavior is unique only to the South Shore Ladder at John Day, it has logically been attributed to the environment within that ladder.

Factors within the fish ladder environment that provide the stimuli for this behavior have been studied extensively over the years by biologists.

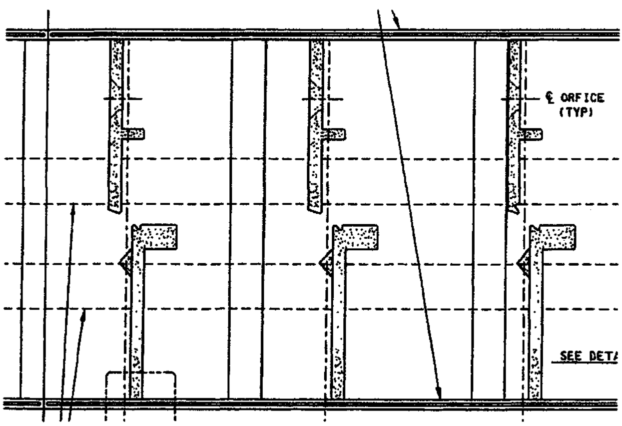
Upon investigation it was determined that changing hydraulic conditions by modifying the control section to a slot and orifice type design would likely solve the problem. To identify the cause of the hydraulic condition, and to develop corrective modifications, a hydraulic model was constructed using a slot and orifice weir design similar to fish ladders at the USACE's The Dalles and Ice Harbor projects. The model allowed visual observation and recording of velocities and directional currents through the control section, and allowed experimentation with weir configurations to determine the hydraulic and biological criteria.

The corrective measures called for construction of a new reinforced concrete weir system within the existing South Shore Ladder. All of the existing reinforced concrete weir walls within the control section were removed, and new cast-in-place reinforced concrete weir walls were constructed within a three-month in-water work period. Each of these new weirs has a vertical slot and an orifice, including a mechanical gate/sill at the base of some of the slots to control flows at different forebay elevations. The existing weir walls were saw cut for removal, the new weir walls was drilled and grouted into place. A total of thirty-nine weir walls were removed and replaced with twenty-three new weir walls with the different geometry (see Figure 4.5-4).

John Day South Shore Fish Ladder
Weir Reconstruction



Weir Walls Before Modification



Weir Walls After Modification

Figure 4.5-4 John Day, Fish Ladder Weir Replacements (courtesy Portland District, USACE)

After construction of the modified weir walls was completed, USCOE hydraulic design personnel verified that the weir walls were performing as predicted in the hydraulic model. Then, when anadromous adult fish started to ascend the ladder, project biologists observed the ladder on a daily basis to see whether or not fish were exhibiting jumping behavior. After a couple of fish passage seasons, fish ascending the ladder no longer exhibit the jumping behavior, so the modifications to the ladder were deemed a complete success.

No. 5 Energy Loss to Fish Attraction Flows
Woodland Project (Kleinschmidt Associates, 1992)

While this Case History describes how energy is recovered for a downstream fishway, the method is also applicable for upstream fish passage facilities.

The Woodland Hydroelectric located in Woodland, ME was constructed in 1910. The project generates 9 MW at 3,000 cfs under 44 feet of gross head. Power generated is used to operate the site's associated paper mill. The project has a Denil fish ladder that has been in operation since the early 1960s to passalewife and Atlantic salmon. Prior to 1991 passage of juvenile fish downstream was through the turbines or the fish ladder. In 1991 a separate facility was constructed for downstream fish passage.

For each fish passage facility, upstream or downstream, there are associated attraction flows dictated by regulating agencies responsible for reviewing the design of a fishway. The purpose of the attraction flow is to create a suitable flow field in the vicinity of the entrance to the upstream or downstream fishway, to attract fish into the system. Although specific site requirements may vary, typical attraction flows required in the eastern United States are 2 to 3% of station hydraulic capacity for an upstream fishway and 2% for a downstream fishway.

Flows to transport fish through the fishway are a part of the flow that is used to attract fish except the transportation flows are often significantly less than the required attraction flow. Because of costs to transport fish upstream under high flows, it is not common to construct a fishway to transport fish in the attraction flow. For upstream fishways the simplest method of supplying the "supplemental" attraction flow is to augment the transportation flows directly from the headpond through a secondary water pipe. For downstream fishways the simplest method is to draw the total volume of the attraction flow into the entrance then transport these flows around the dam and discharge them into the tailwater. While the simple methods minimize the civil cost of the fishway, there is a loss of annual generating revenue in the discharge of the "supplemental" attraction flow. A small hydroelectric turbine could be used to generate power using the additional attraction flow if the volume of flow and the head at the site is sufficient to economically justify the costs.

A method of recovering some of the energy potential associated with the discharge of the attraction flow is to pump the supplemental flow requirements out of the tailrace or impoundment. The pumping can be done with inexpensive submersible, low head, mixer pumps with low energy demand. For example, if the supplemental flow is pumped against one foot of head at a 50% pump efficiency, the total loss to net station generation from operating the fishway is equal to the transport flow at full station net head plus the supplemental attraction flow at two feet of head and the power consumed to operate the pump. In many instances the reduction in lost generation can pay for the additional equipment and costs associated with pumping in the first couple of years of operation.

Figure 4.5-5 shows the general arrangement of the pump-back system that was constructed at the entrance to the Woodland downstream fishway. The total attraction flow was 60 cfs, of which 20 cfs was required to transport fish. In addition to the infrastructure that was to be constructed for the fishway, the pump-back system required the installation of a 40 cfs mixer pump operating at a 1.5 foot head, discharge cone, check valve, a second slide gate to control flows to the pump, controls, and fish screen and support frames to prevent fish entrainment into the pump. The incremental construction cost for the pump-back system was $38,000.

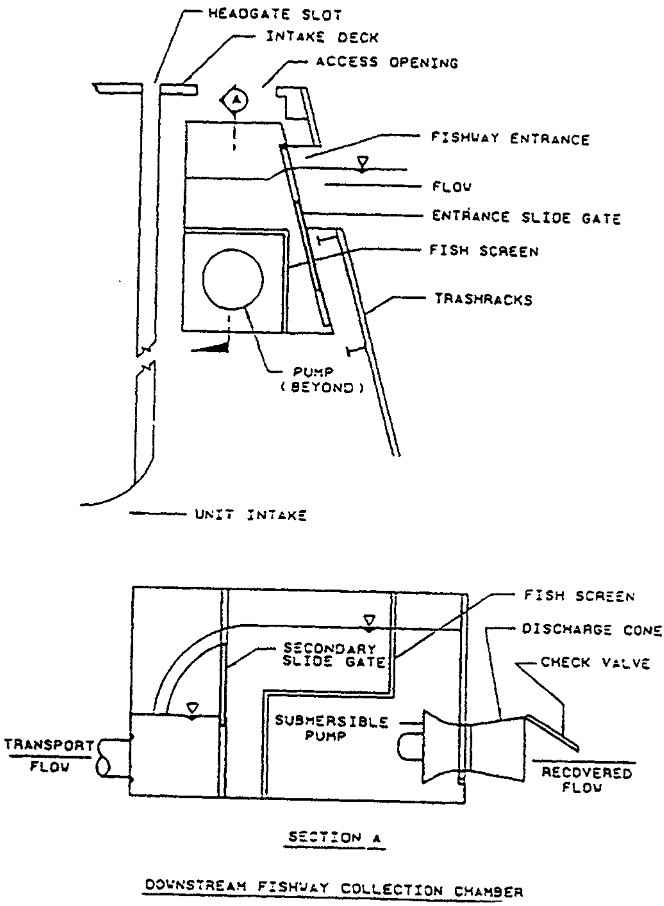


Figure 4.5-5 Attraction Flow Pumping System (courtesy of Kleinschmidt Associates)

The downstream fishway operated 244 days a year. Power consumed by the pump-back system in "recirculating" 40 cfs was 15 kw, for an average annual energy requirement of 88 MWH. The 40 cfs that was not discharged as attraction flow was used to generate 112 kw of power, for an average annual generation of 510 MWH. The pump-back project provided a net annual benefit to the project of $25,300 (1992 dollars).

No. 6 License Mandate for Fish Passage Greenville Dam (Kleinschmidt Associates, 1996)

The Greenville Dam project, located on the Shetucket River, is the first dam on a river currently undergoing anadromous fish restoration. The Greenville Dam is a 16 ft high granite capped timber crib dam constructed around 1850. The hydroelectric project generates 2.2 MW under a gross head of 17 ft and 2,120 cfs. The project includes two powerhouse on a 3,200 ft long canal.

When the Owner obtained a new license from the FERC for the continued operation of the hydroelectric project, reviewing agencies (USFWS and Connecticut Department of Environmental Protection) requested, as a condition of the license, that an upstream fish ladder be installed to provide access to the watershed for upstream migrating American shad, Atlantic salmon, alewife, blueback herring, and sea-run brown trout.

The cost of the proposed fish ladder was substantial, in part driven by an agency requirement that the fish ladder needed to be designed to handle a projected fish run size which the Licensee felt was unrealistically high. During the final design process, the governing agency significantly reduced the projected size of the fish run, which would have allowed the construction of a smaller, more economical climbing steep pass ladder, but the timing of the change occurred too late in the design process to be incorporated.

The upstream passage was to be designed as an unmanned facility whose tailwater was impacted by tides (normal fluctuation of one foot, but up to seven feet during spring flows), and which would not result in any increase in the upstream flood level as determined by FEMA. Additional challenges included getting state and federal agencies in agreement. The FERC License established the requirements but there was conflict between the FEMA and state environmental agencies which affected the cost of the project.

To reduce project costs the Owner proposed and constructed a fish elevator, sized to pass the entire project migration run. The lift automatically cycles once every four hours until the run size required more frequent operation. In this way the fish lift can accommodate the Owner and agency expectations of run sizes, while costing significantly less (30%) than the original agency proposal.

The fish elevator consisted of a single 8 foot square hopper with entrance and discharge gates, and an attached floor grillage to crowd the fish into the hopper as the hopper is raised. The hopper, fill valves, entrance gates and discharge gates all operate automatically. The facility traps and discharges fish using a flow of 30 – 60 cfs with additional flows provided at the entrance to the facility to increase the attraction discharge to 100 cfs. The facility includes a room wherein State personnel can count and identify the type of fish moving through the elevator, and the means for those personnel to shunt selective fish to a containment area for removal and examination or for trap and truck operations for relocating fish to other rivers.

The work also included the construction of a downstream fish passage facility located in the power canal just downstream of the fish elevator. The facility consists of a rack structure angled across the power canal with automated trash rake and trash sluicing system, gated entrance chamber, and transport pipe. The rack structure has a clear spacing of one inch between the bars and is used to redirect the migrants from the canal back to the river downstream from the main dam, using a transportation flow of 50 cfs. The locating of the rack structure at the head of the canal was more economical than constructing similar provisions or making modifications to the intakes at each of the project's two powerhouse.

No. 7 Dam Repair and Mandate for Fish Passage
Rainbow Dam (Kleinschmidt Associates, 1993)

The Rainbow Hydroelectric Project, located in Windsor, CT, was constructed in 1925 to provide electric power to a local industrial manufacturing plant. The dam has a structural height of 58 feet and is used to develop the site's 60 feet of gross head for the generation of 8 MW by two turbines which had a combined hydraulic capacity of 2,500 cfs. The Project is not regulated by the FERC but is subject to Dam Safety Regulations of the State of Connecticut.

In 1988 it was determined that the 400 foot long by 52 foot high concrete gravity spillway required resurfacing to restore the downstream face of the dam and to increase the dam's stability. The dam had lost six to eight inches of concrete to freeze-thaw action, and water erosion over nearly all of its downstream surface.

As a condition of obtaining the necessary permits from various state and local agencies to allow the resurfacing work to be performed and to obtain permits to allow the construction of a temporary road across a normally dry riverbed to access the dam, the Owner was required by the State to install downstream fish passage. State environmental agencies argued that under state regulations the proposed work constituted a change in project geometry [although there was no change in the dam's length, height, or discharge capacity] and therefore downstream fish passage could be perscribed and made a condition of the permits. Upstream fish passage had been installed at the site in the 1974 by the State for the passage of Atlantic Salmon and American Shad. Downstream passage had been attempted in 1978 by way of the existing ladder, but the fish could not successfully descend the ladder.

A simple but economical fish by-pass arrangement was initially proposed, consisting of two overflow sluice gates and a 36-inch diameter pipe to be located on the spillway side of the powerhouse, with the fish being discharged directly into the powerhouse tailrace. This fish passage arrangement would cost less than $75,000 to construct and would be installed as part of the dam resurfacing, with the discharge pipe encased into the new resurfacing concrete. The arrangement would make use of an existing trash sluice gate, which would be replaced by the two new gates placed in series to allow the discharge of either trash and debris or the downstream passage of fish.

The proposed arrangement was not acceptable to the agencies as it did not allow them a means to trap fish for scientific-biological study. Via the permitting process, the agencies mandated that the downstream fish passage was to include a sorting facility and provisions for automated monitoring of fish type and numbers, along with a holding facility for conducting scientific-biological study.

The facility that was constructed cost $450,000 in 1993, and included the fish entrance, migrant transportation pipe, sorting chamber, and return pipe to the powerhouse tailrace. The (permanent) holding facility was not constructed, although a final design and construction documents were prepared.

The fish passage facility was designed to accommodate attraction flows of 30 to 60 cfs under a head fluctuation of two feet. The facility utilized an existing sluice gate and trough system that was (and is) used to sluice debris removed from the trashracks. The entrance and attraction flow to the downstream fishpassage facility is controlled by two 6 ft by 6 ft motorized overflow gates located in series. The upstream gate is the entrance gate and is used to maintain the attraction and transportation flow to the fish passage facility. The entrance gate is automated and controlled by a pond level controller. The second gate is manually controlled and installed directly across from the entrance gate and when raised allows fish passage and when lowered allows trash sluicing.

After passing through the entrance gate, fish proceed through the existing trash trough towards the downstream migrant pipe. The trash trough is located on the downstream side of the trash racks, and is still used for sluicing trash. Grizzly racks were installed in the trough to prevent large debris from entering the migrant pipe, and two small gates were installed to allow backwashing of the trough between the trash sluicing and fish passage operations. The fish are transported 400 ft via the migrant pipe to the sorting chamber. The migrant pipe varies in size from 24 to 36 inches and is constructed of steel and high density polyethylene. The pipe operates under super-critical flow due to the layout of the site, a condition which allows the reduction of the hydraulic head at the sorting chamber.

The sorting chamber, essentially a concrete flume with a wedge-wire screen inside, is used to separate the fish from the water for purposes of counting or shunting to scientific study stations. Fish enter the sorting chamber and ascend a screen which is installed on a permanent 15 degree upward incline. The fish transport flow is drawn off beneath the screen and discharged into the migrant pipe which returns the fish to the tailrace 100 ft away. The water in the sorting chamber must be maintained at a near constant level. With the attraction flows varying from 30 to 60 cfs, the water level is maintained by a 3 ft high by 4 ft wide weir gate inside the sorting chamber and located below the screen.

As fish move up the screen they pass over a platform in a shallow depth of water (a few inches). The fish can be video taped as they pass over the platform, allowing the State's fishery personnel to count and identify the type of fish being discharged. At the platform level is a pneumatically operated (shunt) gate that is used to direct the fish to a pipe to the scientific (future) study facility or to the downstream migrant pipe for discharging into the tailrace. The sorting chamber weir gate and the shunting gate are designed to be automated to allow unattended shunting of the fish to the study facility or to the tailrace. Currently fish are shunted to a temporary holding tank and tables which are set up whenever examination of the fish is to be performed. Fish released from the sorting table and holding tank re-enter the migrant transport pipe at the downstream end of the sorting chamber and are discharged into the tailrace.

The scientific-biological study building (future) is to be located next to the sorting chamber. The building would contain a number of holding tanks and sorting tables that will provide the biologist with security for the fish, the ability to perform their examinations out of the weather, and with minimal handling of the fish. The study building may be constructed by the State in the future.

As part of the downstream fish passage work, modifications to the trash racks were also required to prevent entrainment of the downstream migrants. The tops of the existing trash racks are 7 ft below the water surface, and the State agencies mandated that an additional 6 ft of racks be blocked to prevent fish entrainment. A energy study was performed to determined the economic impact, and it was determined that while there may be a slight increase in head loss, it would not represent a significant reduction in power output, therefore steel plates were attached to the downstream side of the upper six feet of the trashracks.

4.5.6 Collective Knowledge

Standards for use in "sizing" or "designing" a fish passage facility are limited. Sizing and siting criteria are usually mandated by a governing environmental resource agency; while standards for use in the design of the civil works, *i.e.* the concrete and steel, are those published by the AISC, ACI, and the USACE. Technical references on Design Criteria and General Information are identified in the following subsections:

a) Design Criteria

Bell, M.C. (1984). Fisheries *Handbook of Engineering Requirements and Biological Criteria. Corps of Engineers*, North Pacific Division, Portland OR, U.S.A. 490p.

Bell, M. C. (1991). *Fisheries Handbook of Engineering Requirements and Biological Criteria.* Prepared for Fish Passage Dev. Eval. Prog., United State Army Corps of Engineers, North Pac. Div., Portland, OR.

Bates, K. M. (1992). *Fishway Design Guidelines for Pacific Salmon.*

Clay, Charles H. (1995). *Design of Fshways and other Fish Facilities.* Boca Raton: Lewis Publishers. 2nd edition. 248 p, ill.

U.S. Fish and Wildlife Service. (1994). Fish Passageways & Diversion Structures Symposium. Holyoke, MA. June 13-17, 1994.

b) General Information

Alden Research Laboratory Inc. (1998). *Evaluation of Fish Behavioral Barriers.* EPRI Report No. TR-109483. Palo Alto, CA.

Alden Research Laboratory Inc. (1998). *Review of Downstream Fish Passage and Protection Technology Evaluations and Effectiveness.* EPRI Report No. TR-111517. Palo Alto, CA, Final Report, November 1998.

Andrew, F. J. (1990). *The Use of Vertical-Slot Fishways in British Columbia, Canada.* Proc. International Symposium on Fishways '90, October 8-10, 1990, Gifu, Japan: p. 267-274.

Bates, K. M. (1992). *Fishway Design Guidelines for Pacific Salmon.*

Clay, Charles H. (1995). *Design of Fishways and other Fish Facilities.* Boca Raton: Lewis Publishers. 2nd edition. 248 p.

Coutant, Charles C. (ed.). (2001). *Behavioral Technologies for Fish Guidance.* Bethesda, MD, American Fisheries Society. 193p. [American Fisheries Society Symposium; 26].

Electric Power Research Institute (EPRI). (1994). *Biological Evaluation of a Modular Inclined Screen for Protecting Pish at Water Intakes.* Palo Alto, CA. [EPRI Report No. TR-104191].

Electric Power Research Institute (EPRI). (1994). *Fish Protection/Passage Technologies Evaluated by EPRI and Guidelines for their Application.* Palo Alto, CA. [EPRI Report No. Tr-104120].

Electric Power Research Institute (EPRI). (1994). *Research Update on Fish Protection Technologies for Water Intakes.* Palo Alto, CA. [EPRI Report No. TR-104122.]

Francfort, J.E. 1991. (1994). *Environmental Mitigation at Hydroelectric Projects.* Washington, DC.: DOE. United States Department of Energy, and Oak Ridge National Laboratory (ORNL)

Franke, G. F. , D. R. Webb, R. K. Fisher, Jr., D. Mathur, P.N. Hopping, P.A. March, M.R. Headrick, I. T. Laczo, Y. Ventiikos, and F. Sotiropoulos, (1997). *Development of Environmentally Advanced Hydropower Turbine System Design Concepts.* Prepared for U.S. Dept. Energy, Idaho Operations Office Contract DE-AC07-94ID13223.

Slatick, E. (1975). *Laboratory Evaluation of a Denil-Type Steeppass Fishway with Various Entrance and Exit Conditions for Passage of Adult Salmonids and Shad.* Marine Fisheries Review, Vol. 37, No. 9.

Stone and Webster Environmental Technology and Service. (1995). *Impacts of Hydroelectric Plant Tailraces on Fish Passage : A Report on Effects of Tailraces on Migratory Fish and Use of Barriers, Modified Project Operations, and Spills for Reducing Impacts.* Washington, DC.: FERC. 101 p.

4.5.7 Technical References

BC Hydro. (1995). Project files, BC Hydro, Burnaby, BC, Canada

Kleinschmidt. (1992). Project files, Woodland Downstream Fish Passage Project, Kleinschmidt Associates, Pittsfield, ME.

Kleinschmidt. (1993). Project files, Rainbow Dam Repair Project, Kleinschmidt Associates, Pittsfield, ME.

Kleinschmidt. (1996). Project files, Greenville Dam Fish Passage Project, Kleinschmidt Associates, Pittsfield, ME.

MWH. (1996). Project files, McNary Dam Fish Passage Hydroelectric Project, Northern Wasco County People's Utility District.

United States Army Corps of Engineers, Portland District. (USACE). (2000). Bonneville Project Files, Portland, OR.

United States Army Corps of Engineers, Portland District. (USACE). (2000).John Day Lock and Dam Project Files, Portland, OR.

4.6 Trash Racks

4.6.1 Function

Trash racks prevent debris that could damage the turbine and other components from entering the intakes. On pumped storage projects, trash racks may also be located on the downstream side of the dam to prevent debris from being pulled into and damaging the pumps during pumping operations.

The rack bars are typically made out of vertical metal bars that are spaced to minimize head loss while still preventing larger debris from passing (see Figure 4.6-1). Normally the metal bars are carbon steel, but stainless and aluminum have also been used. Racks are also constructed of fiberglass or plastic, and have been constructed of wood. The bars are connected together by bolting or welding to form panels that span across the upstream face of the opening to the intake structure.

Figure 4.6-1 Trash Rack at Bonneville Hydroelectric (courtesy of Portland District, USACE)

The clear spacing between the bars can vary from 1 inch to 8 inches. The bar spacing is normally set by the turbine manufacturer, and represents the size of the largest sphere (debris) that can be passed through the runner, without damaging the runner or getting stuck in the nozzles of impulse turbines. Closer bar spacings (one inch or less) are often mandated by environmental agencies to reduce fish entrainment. The rack bars are placed vertically or are sloped (battered) to facilitate the removal of trash.

The rack bars are designed to span in the vertical direction, with intermediate beams providing structural supports for larger spans. Increasing the thickness and depth of the rack bars, and reducing the distance between the horizontal supports can increase the load capacity of the racks. The criteria for designing a trash rack are based on a head differential from one side of the trash rack to the other, as well as ice and debris loading and trash against the racks. The amount of differential head will vary from one third to full head against the racks, based on the height of the trash rack, and the velocity through the intakes. The amount and type of trash will also vary from site to site. In climates subject to freezing conditions and ice formation, the racks and support structures should be designed for full blind over (100% of head differential). Racks and support structures that have been designed for full blind over also provide the project owner with the unique ability to dewater nearly all of the intake by installing sheet pile, plywood, steel plates, or wood planks on the racks to serve as a temporary "cofferdam".

Trash racks for low head conventional intakes are designed with low approach velocities of 1 to 2 feet/second (based on the rack's gross area) to minimize headlosses. On hydroelectric projects with heads on the racks of greater than 100 feet, intakes and trash racks are often designed with approach velocities of 5 to 7 feet/second. On pump turbines, the discharge velocities of older (1960 vintage) plants in the pump back mode may be 15 to 20 feet/second or more, which will cause significant vibration, damage, and possible failure of the racks and/or the support structure. The trash racks on pump turbines must often be designed to withstand hydrostatic and vibratory loads that reverse themselves due to flows occurring in opposite directions for the generating and pumping modes.

The life expectancy of steel trash racks is 15 to 35 years depending on the conditions to which they are subjected, although some projects have had the same trash racks in place for 75 years. The life expectancy of plastic or fiberglass racks is theoretically unlimited from the corrosion perspective, but generally considered to be 25 to 50 years due to physical damage to the racks over time. The supporting steel has a life expectancy of 75 or more years, depending on the performance of routine maintenance and the chemical composition of the water. Due to oxidation, vibration and exposure to a fluctuating waterline, the area of the trash racks and support steel most susceptible to corrosion is the area three feet above and below the normal water surface.

4.6.2 Problems

Trash rack problems are generally due to deterioration and increased head loss across the trash racks, caused by the following problems:

- Improper design.
- Structural damage.
- Deterioration.
- Debris and trash accumulation in front of the racks.
- Frazil ice.
- Anchor ice.
- Vibration.
- Zebra mussels.

Ineffective or deteriorated trash racks could allow large debris or parts of the trash rack to enter the water passageway and possibly damage the turbine and ancillary equipment.

4.6.3 Corrective Measures

For the problems identified in Section 4.6.2, the following are possible corrective measures.

a) Design

Shallow intakes with poor approach flows may result in the formation of vortices, causing vibration and damage to the racks and support structure. For intakes with an improperly sited, or improper or poorly designed trash rack system, there are few prescriptive solutions to correct problems. The problems must be addressed specifically for the site, and a cost effective solution may be identified by constructing and running tests on a model (physical or possibly computer) of the intake structure.

To reduce trash accumulation and improve hydraulic efficiency, it may be possible to increase the bar spacing. Bar spacing is based on several variables, as the bars need to be close enough to prevent debris from passing, and wide enough to minimize head loss.

b) Structural Damage

Any bent or deteriorated bars or support members that may be restricting flow and causing a differential head from one side of the trash rack to the other should be repaired or replaced. Coating the steel bars with epoxy coatings may help keep the bars from deteriorating. While bars with paint, epoxy, galvanizing or other coatings may be used to reduce corrosion, the use of protective coatings is often defeated by damage caused by the trash raking equipment. An inspection of the racks on a routine basis is warranted to monitor their condition.

c) Deterioration

Deterioration of the racks and support members may be addressed by simply replacing the deteriorated members. If deterioration is being accelerated by poor or aggressive water quality, then an evaluation should be made to assess the suitability of materials used for the trash racks and support members.

d) Debris

With an intake equipped with rack rakers, an accumulation of debris in front of the racks may be caused by an undersized, poorly designed, poorly operating or operated trash rake, or it may be necessary to operate the trash rake more frequently. Large debris often sinks and will accumulate along the bottom the racks if the trash rake does not have the capacity to reach the bottom of racks. It is not uncommon for the lower one-third to one-half of the racks to be completely obstructed at sites with insufficient means to remove the debris. The installation of an effective log boom may help to deflect large debris away from the intake, thereby reducing the accumulation of smaller debris on the racks.

Racks may periodically require cleaning to remove debris from between the bars. The cleaning is often done using high pressure water, either underwater or in the dry.

e) Ice – Frazil

If ice build-up is an accumulation on the racks from floating pieces, as occurs during spring breakup, trash raking is often the only removal solution. If frazil ice occurs there are a number of options that may work:

i. Install air bubblers (or water circulating pumps) in the water at the bottom of the racks, deep enough to provide a thermal change of water temperature.

ii. Temporarily cease generating operations, or reduce the approach velocity of the water to less than 2 to 3 feet per second to allow frazil ice to form a stable ice sheet to help insulate the water. Frazil ice usually forms in the early winter after an intense cold spell, when the water is super cold, but before ice has sufficiently formed on the surface of the impoundment, and often when there is an arctic wind blowing across the open water. Often the conditions are "ripe" for the formation of frazil ice for only a short duration of a few hours or a couple of days.

iii. Modify the trash racks to break their thermal conductivity. Metal racks projecting above the surface of the water transmit temperatures below freezing into the water, promoting the formation of ice on the racks. Installing non-conductive racks (plastic or fiberglass), or using racks that are fully submerged usually resolves the problem. If the metal racks must project above the water's surface, providing a physical non-thermal conducting break in the racks just below the water surface may reduce the extent of ice formation on the submerged portion of the racks. Electrically heating of the bars has been done and patents exist for bars that are shaped to allow heating, although the cost of heating the bars has not proven to be either effective or economical.

iv. A trash boom can be used to help reduce water surface velocity allowing frazil ice floating at or near the surface to accumulate and form sheet ice in front of the boom.

f) Ice – Anchor

Anchor ice generally forms from the bottom of the racks and grows upward. This problem is generally associated with intakes located on rivers with swift moving water. Ice formation occurs on the riverbed and bottom of the racks when the river slows rapidly. It is not always possible to remove the ice using mechanical trash rakes. Anchor ice is best prevented from accumulating by providing a thermal discontinuity or break in the racks.

g) Vibration

To reduce vibration, deteriorated concrete or structural steel support members not adequately supporting the racks should be repaired or replaced. Excessive vibration of the trash racks may be caused by missing anchor bolts, bent or damaged bars, deteriorated concrete, excessive support spacing. Vibration is most often caused by irregular flow due to eddies or vortices and distance from turbine and rotation. Vibration due to hydraulic conditions may be due to improper design (see Section 4.6.3.a).

The vibratory problems associated with trash racks on pump turbines are beyond the intent of this guideline. Detailed design and operational information for trash racks on pump turbines have been developed by the American Society of Mechanical Engineers (ASME, 1996).

h) Zebra Mussels

Zebra mussels attach themselves to the trash racks, reducing the clear openings between the vertical bars. Because the organisms are often located between the vertical bars, they are not dislodged by mechanical trash raking equipment. To prevent a build up of zebra mussels, chemicals or plastic, fiber reinforced racks, can be used. Special coating can be applied to steel bars to prevent the zebra mussels from sticking. If the problem is not significant, periodic cleaning of the racks with high-pressure water may be sufficient. Other methods of controlling zebra mussels include acoustical vibration, electrical current, thermal shock treatment, ultraviolet light, and CO2 injection.

4.6.4 <u>Opportunities</u>

The main opportunity in repairing or upgrading trash racks is that it may help reduce head loss, thereby increasing power generation, and reduce the operating cost of raking trash. Another opportunity, when replacing components, is to upgrade the racks using corrosion resistant materials.

4.6.5 <u>Case Histories</u>

No. 1 Deteriorated Steel Trash Racks
Berrien Springs (AEP, 1996)

The Berrien Springs Project is located in Michigan on the St. Joseph River. The project was built in the 1920's. The trash racks at the Project were constructed at a 15 degree incline with the rack sections sitting in intermediate steel supports. The trash racks total 190 feet in length with a height of 24 feet with ¼ inch x 3 inch bars spaced at 2 inch center to center. The depth of water at the racks is 21 feet.

The existing trash racks were badly deteriorated and had become a structural concern. A secondary problem existed because the racks were not bolted to the supports. As a consequence, the trash rake would catch on the rusted section of racks pulling them out of their slots which would require bringing in divers to reposition them.

New trash racks were installed with J bolts, which secured the sections of racks to the steel support members preventing the trash rake from moving them. The benefit for replacing the trash racks was that it allowed the continued reliable operation of the facility and the new J-bolts eliminated the need to bring in divers to reposition the racks.

No. 2 Temporary Trash Rack Repair
Oswego Falls East (Brookfield Power New York, 1999)

The Oswego Falls East Hydroelectric facility had a submerged steel trash rack structure. The racks were 27 feet wide by 24 feet high for each of the three units with ½ inch x 3 ½ inch bars spaced at 6 inch center to center.

Portions of the trash rack had deteriorated and were starting to deform and become unsafe. To temporarily correct the problem, steel trash rack panels were fabricated and installed on the downstream side of the existing trash rack structure in existing stop log slots located at the upstream face of the powerhouse. The temporary racks were designed to screen out large debris in the water and prevent any steel members that could fall off the existing trash rack structure from being carried into the turbine. In addition to the chosen corrective measure, the other alternative considered was a complete replacement of the existing trash racks and support structures. Since funds were not available in the current year to accomplish the complete replacement, the owner decided to make the temporary modifications until such time as funds become available to replace the deteriorated structure. The new permanent trash racks will eventually be replaced in kind with the exception of having a closer bar spacing, required by a recently issued FERC license, to reduce fish entrainment into the turbines. Although various alternate locations were considered, the original location is preferred because it maintains better fish protection due to the angled nature of the structure.

The corrective measure provided the owner with a low cost temporary fix which allowed the facility to continue to generate until funds became available to fully replace the deteriorated trash racks and support structure, which subsequently occurred.

No. 3 Trash Rack Failure Due to Inadequate Design
Barge Canal Hydraulic Race (Brookfield Power New York, 1998)

The Hydraulic Race Hydroelectric Facility obtains water flow from the NYS Barge Canal in Lockport NY and discharges into 18 Mile Creek. The facility was constructed in the 1920's. The hydraulic race is a tunnel from the canal to the hydroelectric plant. The trash racks are positioned at the mid-tunnel point. The racks are comprised of an upper section and a lower section 12 feet - 5 ¼ inches wide. The lower section is 26 feet x 1⅛ inches high with ½ inch x 4 ½ inch bars spaced at 5 ⅜ inches center to center. The upper section is 8 feet x 2⅜ inches high with ⅜ inch x 3 ½ inch bars spaced at 5 ⅜ inches center to center. During the period of mid November until late April every year the adjacent section of the NYS Barge Canal is drained and therefore all operations at the hydroelectric plant cease. The trash rack structure had been replaced at this facility in 1995.

In March of 1998 a routine inspection discovered that the new trash rack supports had failed. The design of the replacement rack supports utilized relatively light framing based on the fact that the facility is never subjected to icing which could cause the rack to become 100% clogged. Fully clogged or "blinded" racks subject their supports to full hydrostatic head. The design was based on no more than 25% blinding resulting in only a small reduction of headloss. During the subsequent investigation of the rack structure the engineers discovered suspicious amounts of heavy blue plastic that was imprinted with the figures of fish and other sea life, in other words, a pool liner. It was concluded that a pool liner had been dumped into the canal and had made its way into the intake tunnel. When this material became lodged against the racks, the subsequent hydraulic differential caused the support to become overloaded and caused the structural failure.

The trash racks and support structure were replaced using a design based on the possibility of 100% rack blinding despite the lack of winter operation, which allowed continued reliable operation of the facility. The replacement of the structure provided the opportunity to improve the design of the racks to withstand increased load and to allow the continued reliable operation of the facility.

No. 4 Blockage by Ice and Zebra Mussels
School Street Hydroelectric (Brookfield Power New York, 1990)

The School Street Hydroelectric Project is located on the Mohawk River, in New York and was built in 1915. The project consists of a 1278 foot long gravity dam, a 5100 foot long earthen intake channel with an upper and lower gate house, and a five unit powerhouse. Adjacent to the dam is an ice fender structure, which deflects ice and floating debris from entering the intake channel through the upper gatehouse structure. The upper gatehouse structures does not have trash racks. The lower gate house has an intake area 24'- 6" high by 121'-0" wide with trash racks. The steel rack bars are ½" x 4" spaced at 3" center to center.

The existing steel trash racks had two basic problems. The first problem was the racks were susceptible to a build-up of frazil ice in the winter, which adhered to the bars and continued to build up until it completely closed off the openings between the bars. This resulted in considerable down time of the generating units each year to clear the racks. The second problem was zebra mussels attaching to the racks also requiring outages to clean the racks.

Although several alternatives were examined including rack cleaning devices, which required outages to clean the racks, the most economical alternative was to replace the existing steel trash racks with plastic racks. The benefit to the owner for replacing the steel trash racks with plastic racks was increased revenue by eliminating the need to take generating units off line to clean frazil ice and zebra mussels from the racks.

No. 5 Trash Rake Failure Due to Inadequate Design (Pump Turbines) Smith Mountain (AEP, 1967)

The Smith Mountain Pumped-Storage Project is located in Virginia on the Roanoke River. The Smith Mountain Dam is a concrete thin arch structure with a maximum height of 237 feet. Construction of the dam was completed in 1965 at which time; there were four generating units, two pump-turbines and two conventional turbines. In 1979 a fifth unit (pump-turbine) was installed.

Trash racks were installed on the upstream side of the dam, and on the downstream side of the powerhouse. The intake racks were cage like structures, attached to the upstream face of the dam. The cage consisted of a frame of structural steel shapes and the racks were bolted to the frame.

After the initial start-up of the project in 1965, the trash rack sections for unit No. 1 were found laying in the forebay during a diving inspection. Unit No.1 was the first pump-turbine unit to become operational. The racks were literally blown off the support frame. Prior to making any repairs, to establish the appropriate design criteria for the modifications a physical model testing was performed to determine the velocity in and around the trash racks under both the generating and pumping modes of operation. As a result a diving contractor was brought in to reset the racks using reinforced connections. Field measurements of the velocities at the trash racks when operating in the pumping mode verified the predictions of the physical model. But, the diving inspection following the next start-up found both the trash racks again blown off the support structure. Due to the complexity of the turbine penstock geometry, its relationship to the upstream face of the dam, and other hydraulic disturbances due to the orientation of the two "intakes" with the discharge channels, the 1965 physical model was not suitable to replicate the von Karman vortices and predict the associated forces.

The main cause of the problem was that while the design addressed the forces associated with the velocity of discharge when pumping, the design did not take into account the high forces from the turbulent pump outflow which resulted in a significantly under designed structure. The trash rack and support frame were not designed to present day hydraulic standards, as the intake is the same diameter as the penstock without benefit of a transition that expands to the width of the trash racks. The absence of the expanding transition prevents the flow velocities during pumping from reducing as the flow exits the penstock intake. The discharge velocities at the trash racks were determined to be as high as 20 fps as indicated by the model and field testing.

In 1967, shortly after the second failure of the trash racks, a removable trash rack system was built. The trash racks were designed based on the criteria derived from the physical model testing. The trash racks were mounted on a steel frame and a hoist was installed to allow the front (upstream most) racks panels to be raised during the pump mode and lowered during generation. The structural support frame was left in placed, but because of the reduction of structural continuity due to the removable racks, eventually failed and required additional bracing. The bracing also experienced problems requiring future modifications by adding an anchor beam at the bottom of the bracing.

4.6.6 Collective Knowledge

For design aids or design references, see:

1. American Society of Civil Engineers (ASCE). (1995). *Guidelines for Design of Intakes for Hydrolelectric Plants.* ASCE, New York, NY.
2. American Society of Mechanical Engineers(ASME). (1996). *Guide to Hydropower Mechanical Design: Chapter 6.* ASME, New York, NY.
3. Creager, W.P. & Justin, J.P. (1950). *Hydro-Electric Handbook.* 2nd edition. John Wiley and Sons, New York, NY.
4. Sell, Lloyd E. (1971). "Hydroelectric Power Plant Trashrack Design" *in American Society of Civil Engineers, Proceedings, January.*
5. United States Army Corps of Engineers (USACE), Cold Regions Research and Engineering Laboratory. Hanover, NH.
6. United States Bureau of Reclamation (USBR). (1987). *Design of Gravity Dams.* USBR, Denver, CO.
7. Zowsky, T. (1960). "Trashracks and Raking Equipment" in *Water Power,* September.

4.6.7 Technical References

American Electric Power (AEP). (1996). Project files for American Electric Power, Columbus, OH.

American Electric Power (AEP). (1967). Project files for American Electric Power, Columbus, OH.

Brookfield Power New York. (1999). Project files for Brookfield Power New York, Liverpool, NY.

Brookfield Power New York. (1998). Project files for Brookfield Power New York, Liverpool, NY.

Brookfield Power New York. (1990). Project files for Brookfield Power New York, Liverpool, NY.

4.7 Trash Rakes

4.7.1 Function

The purpose of the trash rake is to remove debris from in front of the trash racks. Removing the trash will reduce head loss across the trash racks and improve the generating efficiency of the turbines.

There are several types of trash rakes ranging from manual to automated mechanical systems. A manual system consists of a person using a long handled rake removing the debris by hand. Automated systems can be programmed to run on differential pressure across the trash rack, at timed intervals or on manually based on visual observation.

There are several different styles of automated rakes including: fixed-dragline (non-crane type), dragline (mobile crane), hydraulic (boom or telescoping), hoist and carriage, and backhoe type. The fixed-dragline rake consists of a beam attached to cables. The beam is pulled out several yards in front of the intake, pulled along the river bottom, and then pulled up the trash racks. This type of rake helps to prevent siltation of the forebay, along with removing debris for significant distances in front of the trash racks. Other styles of trash rakes either have a raking arm attached to cables (hoist and carriage), or a hydraulic arm, which lowers the raking head to the bottom of the trash rack and pulls it up, removing the debris. Some rakes are set up to dump the debris into a trough or sluiceway which allows the trash to pass downstream without the need of handling anything except logs and man made debris. Other rakes have a bin attached to them to rake the trash into. The trash is then removed from the bin for disposal.

The fixed-dragline type rakes are simple economical machines, and provide the ability to clear debris the greatest distance upstream of the intake racks. Most boom type, and hoist and carriage type, trash rakes have reaches that extend no further than one to five feet upstream of the base of the trash racks. The cleaning reach of a dragline is limited only to the distance that the dragline sheaves can be located upstream of the trash racks. The draw back to fixed-dragline trash rakes is that they are normally fixed in place machines, with a maximum raking width of 45 to 50 feet, therefore requiring multiple machines to clean wide trash racks. Dragline cranes have been used as trash rakes, but they are not considered to be as effective as a rake that operates parallel to the flow of water.

Siltation of an intake occurs when twigs and branches, mixed with leafy material, are not effectively removed from the trash racks, allowing sediments to accumulate, which in turn promote further accumulation of debris and sediment (a vicious cycle). As the deposits build up, the trash rake may not be able to reach past the material to allow removal. Deposits have been known to reach depths of twenty feet or more. If the twigs, branches, and leafy material are removed, the sediment can then pass through the racks and be discharged by the turbines.

Most trash rakes should be able to handle leafy materials, but this capability varies. Some machines may not be able to handle large amounts of leafy material or floating vegetation mats, while other rakes have the capacity to lift loads of 2,000 to 5,000 pounds as associated with logs. Many trash racks can be automated, with the automation systems being the most complex for those machines that travel along the length of the trash racks. Automated rakes need either an automated or manual means to remove debris that has been pulled from the trash racks. All trash rakes have some weakness when operated in freezing climates, and there is no single type that will outperform another under freezing conditions. Handling of debris removed from the racks is also a significant problem in freezing climates, and there is no good solution other than using large mechanical equipment such as loaders and dump trucks. Ideally, if a site could be equipped with a trash sluice (trough or gate), then it may be possible to discharge the debris downstream without need to remove it from the river. If the dam owner has a number of dams on the river, then consideration should be given to the effort and costs required to handle and discharge debris downstream. It may be more economical to remove the debris at the upstream site, thereby eliminating the need to handle the material a number of times.

When trash racks are hand raked, the racks are inclined at a rate of 1H:3V, or flatter, to accommodate the limitations of the human raking ability. Hand raking is limited to only cleaning the racks 6 to 8 feet below the water surface. When using mechanized trash rakes, the racks can be vertical, but are often installed at 1H:10V, or slightly flatter, for the convenience of the installation of the trash racks, and to allow the trash rake head to lie against the racks to promote tracking and minimize dropping of debris. When designing the trash racks, care should be exercised to minimize any discontinuities in the plane of the racks to prevent the rake from "snagging" the racks or dropping its debris. When using mechanical rakes, the trash racks need to be mechanically fastened to the support structure to prevent the rake from shifting the location of the racks.

There is no single type of trash rake suitable for use at all hydroelectric projects. Prior to selecting a particular type of rake or manufacturer, the owner needs to consider: the physical location of the machine, the type of trash to be handled, and the complexity of the design and system used to run the trash rake.

4.7.2 Problems

Common problems and causes associated with trash rakes are as follows:

- Inability to maintain clean racks.
- Reduced efficiency of trash removal caused by a damaged raking arm, or the trash rake being worn or out of alignment.
- Damaged cables or pulleys.
- Worn electric motor or wiring.
- A manual trash raking system which needs to be upgraded to an automated system.

- Disposal of debris removed by the trash rake.
- Inadequate lifting capacity to lift heavy debris or inability to reach the bottom of the trash racks.
- Worn or misaligned rails.

4.7.3 Corrective Measures

For the problems identified in Section 4.7.2, possible corrective measures include:

- Repair or replace damaged components, cables, rails, pulley mechanisms, or electrical components.
- Upgrade to an automated system. The selection of the type of trash rake to install is dependent on the physical room available and the layout of the intake deck. If there is limited space the trash rake may have to extended out over the trash racks.
- Add a trash sluice to allow the natural debris to pass back to the river.
- Upgrade the capacity of the trash rake to a higher lifting capacity to remove heavier debris.

4.7.4 Opportunities

The main problem with trash removal is that it can be labor intensive. All improvements or upgrades to the trash raking system that can help reduce costs and improve generation output should be considered.

4.7.5 Case Histories

No. 1 Debris Handling
Racine Project (AEP, 2001)

The Racine Hydroelectric Project is located on an Army Corps of Engineers Lock and Dam on the Ohio River. The project used a crawler crane with a clamshell bucket as the trash removal system to clear 87 lineal feet of 90 feet high racks in 65 feet of water. The majority of the debris being removed was large timber, which was removed and hauled by truck to a landfill.

The main problems with this crawler crane system were that it was labor intensive, had high hauling and disposal costs, and flow through the turbines had to be reduced in order to remove trash.

To resolve these problems an automated trash rake system was installed. The new trash rake travels on rails across the intake deck. The rake head is attached to cables (hoist and carriage type rake), which lower the rake head to the bottom of the trash rack (see Figure 4.7-1). One of the cables attached to the rake head is used to rotate the rake head. When the rake head reaches the bottom of the racks, the cable is released, allowing the rake head to rotate, clamping the trash against the racks as it pulls up the racks, and dumping it into a trough. Man-made debris is removed from the trough and the natural material is sluiced back to the river on the downstream side of the powerhouse. The trash racks are made up of eight, 11 foot wide sections. The rake is programmed to travel up and down one section of the racks twice and then move to the next section, and the process is repeated until all eight racks are cleaned. It takes approximately 30 minutes for the rake to set up and clean all eight sections of rack. The trash rake can be set to run continuously or at timed intervals. The project currently operates the trash rake on a timed interval, adjusting the time based on river conditions and the season.

Figure 4.7-1 Racine Trash Rake (courtesy of AEP)

The biggest problem encountered with the rake is that the designers underestimated the size of the debris being removed. During the removal of large tree limbs, the limbs could extend above the limit switch (shut-off switch) without tripping the machine off. When this happens the machine keeps operating allowing the portion of the tree limbs extending above the switch to hit the upper section of the machine causing damage. This problem was corrected by adding guard plates to the upper section of the machine.

The benefit of installing the new trash rake is that it reduced overall operating costs of raking trash, trash disposal, and lost generation.

No. 2 Antiquated Trash Rake
Byllesby Project (AEP, 1997)

The Byllesby Hydroelectric Project located on the New River in Virginia had an old hoist and carriage style trash rake (four feet wide) and was manually operated. The trash was raked on top of the intake deck where it had to be manually put into dumpsters and hauled away. The trash racks are 144 feet long by 46 feet high and the water is 39 feet deep at the racks.

The existing trash rake was labor intensive and during high water the rake could not remove debris fast enough to prevent the trash racks from clogging, forcing reduction of flow to the turbines and generating output. Another problem at the plant was the depth of the continual build up of sediment and debris upstream of the trash racks was increasing and threatened to completely block off flow to the intake structure and turbines.

To correct both these problems an automated fixed dragline rake system was installed. The drag rake has a steel beam attached to cables, which pulls the beam out approximately 50 feet upstream of the trash racks. It then pulls the beam back toward the racks dragging it along the river bottom keeping silt from building up, then the beam continues to be pulled up the trash racks, dumping the debris in a trough where the man-made debris is removed and the natural material is sluiced downstream of the powerhouse.

There are four separate dragline units, one in front of each unit intake. The rakes can be programmed to run continuously or at timed intervals. The rakes can also be programmed to go out the entire 50 foot distance or only a few feet in front of the trash racks. The Byllesby Project operates the rakes on a continuous basis during high water and for one hour a week during periods of low flow.

The installation of the dragline trash rack keeps the forebay in front of the intake racks free from silting in and eliminates the need to take units off line during high water due to excessive trash accumulating on the trash racks. These benefits of the automated installation are increased generation and revenue, and reduce labor costs for trash raking.

No. 3 Debris Handling
Dalles North Shore (USACE, 2001)

The Dalles North Shore Auxiliary Water Supply is used to maintain flow conditions in the lower half of the North Shore Adult Fishway and to maintain fishway entrance flow requirements. The intake is comprised of four 10 foot wide intake slots, each with a trash rake. Each slot has two trash racks stacked vertically. The total height of the racks is 25 feet. The trash rake is mounted to a mobile crane (clam shell type) and debris is removed from the trash rake onto the spillway deck, then loaded into a truck.

The raking is labor intensive, and must be done during daylight hours because of visibility and safety concerns for operators and riggers.

A new automated trash rake, with a superstructure mounted on rails, was procured. The trash rake (hydraulic, telescoping boom type) is lowered and raised by two opposing hydraulic cylinders (see Figure 4.7-2). An additional hydraulic cylinder is used to force the rake head against the trash rack. A two-man crew is required, consisting of an operator and a truck driver. The rake is self-positioning, and dumps into a debris container mounted on the rake structure. The container is automatically dumped into the same truck the prior system used. The new raking system can be safely operated at night, and allows the AWS to remain in operation during raking cycles.

Figure 4.7-2 Trash Rake at Dalles North Shore (courtesy of Portland District, USACE)

The controls are set so the rake makes one pass automatically across the entire intake. There is a sensor, which measures differential pressure in the intake or the rake can be operated manually. Although there have not been any complaints with the basic operation of the rake, the major complaint has been with the layout. The layout of the access ladder exposes the operator to passing vehicular traffic and the hopper is awkward for unloading into a dump truck.

The benefit for installing the new trash rake was that it reduced the amount of labor required to remove trash and eliminated the safety concerns allowing the rake to be operated a night.

4.7.6 Collective Knowledge

1. American Society of Civil Engineers (ASCE). Energy Division. Committee on Hydropower Intakes. (1995). *Guidelines for Design of Intakes for Hydroelectric Plants.* ASCE, New York, NY.
2. American Society of Mechanical Engineers(ASME). (1996). *Guide to Hydropower Mechanical Design: Chapter 6.* ASME, New York, NY.
3. Zowsky, T. (1960). "Trashracks and Raking Equipment" in *Water Power,* September.

4.7.7 Technical References

American Electric Power (AEP). (2001). Project files for American Electric Power, Columbus, OH.

American Electric Power (AEP). (1997). Project files for American Electric Power, Columbus, OH.

United States Army Corps of Engineers, Portland District. (USACE). (2001). Project files for USACE, Portland, OR.

5.0 CHAPTER 5 – WATER CONVEYANCES

5.1 Introduction

Chapter 5 introduces the perspective of how real problems associated with water conveyances are identified, defined and implemented to extend the life or upgrade a hydroelectric project or dam. Chapter 5 is organized by principal project feature, namely intake canals, flumes and forebay, tunnels, shafts, penstocks, and tailraces. Tables 5.2-1 to 5.5-1, located at the end of this subsection, summarize by feature, the issues covered, opportunities for life extension and upgrade, and case histories.

As for the previous chapter, the approach has been to document numerous means and methods to upgrade, or extend the life of, civil works associated with a project, while acknowledging that the selection of the most promising solution is often very project dependent. These guidelines are written as a reference tool to assist in this selection, and not as a prescriptive design aid. Moreover, these guidelines are based on experience from existing works, and do not purport to cover all issues that could be associated with civil works.

The definition of life extension and upgrade as used in these guidelines is as follows:

Service Life Extension - Activity that extends the life of the civil feature beyond that which would be expected with normal maintenance (make it last longer).

Upgrade - Activity that improves performance of the civil feature beyond the current performance (make it work better). Exchange of a system or component with a similar or different system may not necessarily be considered an upgrade.

As background, Chapters 1 and 2 describe the processes to extend the life and upgrade hydroelectric civil works, and outline the steps to better understand the issues, identify opportunities, and recognize limitations. These chapters also provide insight on understanding the existing conditions and evaluating proposed changes that include identification and selection of the preferred alternative.

Chapter 3 is a review of innovative technologies that cover civil aspects of hydroelectric projects. These technologies have been developed mainly to reduce costs and improve profitability, reliability and environmental performance. Chapter 3 has been written around seven activities associated with civil works, whereas Chapters 4, 5 and 6 are focused specifically on the civil structures, water conveyances and water control devices, respectively.

This chapter describes examples and solutions as a way to illustrate techniques for improving the performance, or extending the service life, of the civil features of water conveyances. Also provided in this chapter is general broad-based information, or "rules of thumb", to allow the reader to assess if, or when, a structure has reached the end of its service life, or if the structure warrants, or is capable of, improvements to its performance.

The common format used in Chapters 4, 5 and 6 starts with descriptions of the function of specific types of civil features, their problems and limitations, and possible corrective measures and alternative solutions for life extension or upgrade. Not all aspects of the features will be discussed fully. Information on each feature is provided by subsection as follows:

a) Function.
b) Problems.
c) Corrective Measures.
d) Opportunities.
e) Case Histories.
f) References.

Function:

A brief description of the function or purpose of the feature is provided. Some civil features may have multiple uses as part of the hydroelectric project. Depending on the actual use and functions of the structure, its expected service life and opportunities for improvement can be determined.

Problems:

Typical problems and limitations (multiple if applicable), and the (root) causes are identified. If the civil feature has an operating system, a description of the limitations of the system is provided. Some problems are presented as a simple list and others are described in detail. General, broad-based "rules of thumb" regarding the service life, design or operational performance of a civil feature are provided as applicable. These are intended to provide guidance as to whether a feature has reached the end of its life, or the feature and its function could be improved.

Corrective Measures:

Options are identified that have been used (or considered for use) to rectify the problems identified. For solutions with multiple alternatives, the pros and cons of the alternatives are discussed. However, the solution identified may be site specific, and may not be applicable to all similar problems.

Opportunities:

Possible opportunities (additional benefits) are identified for upgrading (improving) the civil feature beyond that associated with just addressing a problem or limitation of service life.

Case Histories:

Detailed examples are provided to describe typical solutions that have been used to address "real life" problems associated with that type of feature. In some of the examples the solutions may have extended the service life of a civil feature, improved its performance, or resulted in both an extension of service life and improvement of performance.

Each case history is structured to provide the following information:

- Background to the project and the function of the civil feature (background).
- Problems and causes (problem).
- Corrective measures and selected alternative (solution).
- Opportunities and benefits (results) provided to the owner as a result of life extension or upgrade. Did the corrective measures solve the problem and/or have a positive benefit/cost ratio?

References:

There are three methods used in each Chapter to support the body of knowledge presented in the guidelines.

- Collective Knowledge.
- Technical References.
- General Resources.

Collective Knowledge is a collection of references considered by the guideline authors to be primary references that describe the function, operation, or design of each civil feature. Some of the references also provide information pertaining to inspection and assessment of the feature, problems, causes and possible solutions. The references are located at the end of each section to which they are pertinent.

Technical References are references specifically identified in the text of these guides, designated by name and date (i.e. ASCE, 1995) with the full reference at the end of each section.

General Resources include any background resource that is deemed to have value for broader reference and additional reading. It includes learned societies, government agencies and other sources. The General Resources Library is found in Appendix A.

Table 5.2-1 Intake Canals, Flumes and Forebays

Issues or Problems Covered

- Deterioration of structures
- Reductions in cross sectional area
- Spalling of concrete lining
- Sedimentation
- Water loss through seepage
- Ice collection and pressure
- Debris collection

Opportunities for Solutions, Life Extension and Upgrade

- Slope stabilization
- Coatings on slopes
- Removal of sediments
- Collection device for sediments
- Sealing joints with epoxy or membrane
- Ice booms or ice structure
- Debris booms or structures
- Dedicated breach section
- Repair and replace
- Upgrade operational design

CASE HISTORIES		PAGE NO.
No. 1	Emergency Breach of Canal Embankment (Flood Condition)	240
No. 2	Ice Induced Failure of Forebay Skimmer Wall	240
No. 3	Deterioration of Water Conveyance System	242
No. 4	Upgrade of Forebay and Pipeline	243
No. 5	Plant Expansion and New Forebay Dam	244

Table 5.3-1 Tunnels, Shafts and Other Underground Openings

Issues or Problems Covered

- Hydraulic losses
- Instability of rock
- Leakage – behind liners, through unlined rock
- Surging and transients
- Rockfalls
- Inadequate investigations of geotechnical conditions
- Fault zones
- Deterioration of linings

Opportunities for Solutions, Life Extension and Upgrade

- Lining to reduce leakage or reduce friction
- Rock bolts etc. for stability
- Concrete or steel lining for stability
- Flexible membrane installation
- Replacing a penstock with a tunnel
- Effectiveness of surge chamber
- Grout voids
- Shotcrete lining

CASE HISTORIES		PAGE NO.
No. 1	Structural Instability of Tunnel	249
No. 2	Leakage of Concrete Tunnel Liner	250
No. 3	Inadequate Investigation of Subsurface Conditions	252
No. 4	Fault Zones and Deteriorated Concrete Liner	254
No. 5	Deteriorated Wood Pipeline (Replacement)	255
No. 6	Tunnel Surge	256

Table 5.4-1 Penstocks

Issues or Problems Covered

- Support structural stability
- Leakage
- Hydraulic performance deterioration
- End of service life, failure
- Wall thinning
- Foundation deterioration – settlement, faults, slides
- Ovaling, out of round
- Localized corrosion
- Aquatic growth (zebra mussels)

Opportunities for Solutions, Life Extension and Upgrade

- Apply coatings to exterior to protect
- Apply linings to interior to decrease headloss, inhibit corrosion
- Repair joints, supports, anchors
- High pressure cleaning, relining
- Reduce friction losses
- Replace joints, supports and anchors
- Replace penstock with steel, wood, HDPE pipe

CASE HISTORIES		PAGE NO.
No. 1	Wood Stave Penstock Deterioration (Replacement)	262
No. 2	Wood Stave Penstock Deterioration (Replacement)	264
No. 3	Steel Penstock Deterioration and Failure	266
No. 4	Foundation Movement	268
No. 5	Deterioration and Increased Headloss	268

Table 5.5-1 Tailraces

Issues or Problems Covered

- Erosion
- Deposition – sedimentation
- Deterioration of walls and embankments
- High and low hydraulic conditions

Opportunities for Solutions, Life Extension and Upgrade

- Excavation to lower tailwater level and increase head
- Protection measures to prevent erosion
- Structural raising tailwater level
- Repair of training walls
- Tailwater suppression system

CASE HISTORIES		PAGE NO.
No. 1	Low Season Tailwater Level	275
No. 2	Flood Erosion of Tailrace Embankment	275
No. 3	Deteriorated Training Wall	276
No. 4	High Tailwater Level	277

5.2 Intake, Canals, Flumes, and Forebays

Water conveyance systems for hydroelectric projects generally fall into two categories: those which supply water through free-flow means, and those systems which utilize pressure conduits. Though intake canals, flumes and forebays serve specific and varied roles on a hydroelectric project, they are free-flow conveyance systems and are therefore discussed together within this section. Pressure conveyance systems including penstocks and tunnels are covered in subsequent sections.

5.2.1 Function

Intake canals convey water from its source, a river, or impounded lake for example, to the powerhouse. They are hydraulic channels excavated in earth or rock, which operate under the laws of open channel flow (Doland, 1954). Canals take many shapes and sizes and may incorporate sophisticated lining or no lining at all. For example, the intake canal at the Kootenay Canal in British Columbia is a concrete lined, v-shaped canal whereas the canal at the Taum Sauk Pumped Storage plant in Missouri is an unlined, vertical sided channel excavated directly in rock. Each serves the same function, conveying water to the power facilities, but the two canals' maintenance requirements may vary drastically.

Flumes are essentially small canals formed by man-made structural elements. They consist of an hydraulic channel, usually square, rectangular or semicircular in shape, and are constructed of timber, wood-stave, metal, concrete, masonry or other suitable substance. The flume may be on grade or supported on piles, structural steel, concrete piers, or wood framing. Due to the costs associated their construction, flumes are generally used to convey smaller quantities of water than that provided by canals. The most well known flumes in the world, though not constructed for hydroelectric purposes, are those constructed in ancient Rome. The Roman Aqueducts consisted of eleven major aqueducts (Figure 5.2-1), built between 312 B.C. and 226 A.D., the longest being 59 miles and incorporating elevated flumes over 100-feet high.

Forebays are also considered part of the conveyance facilities, although their main function is to provide limited storage for the power facilities during operational changes. That is, forebays are sized to provide the initial supply water needed when increasing plant output while the water in the conveyance facilities is being accelerated, as well as to accept the rejection, or surplus, water resulting from a decrease in plant output. Forebays may be a separate head pond or may be integral with the intake canal, such as at the Box Canyon project on the Pend Oreille River in the state of Washington (Davis, 1993). Forebays may also be integrated within the water conveyance system, as is the case at the City of Tacoma's Mayfield Powerstation, also in the state of Washington. There the forebay consists of a structure located at the end of the power tunnels but upstream of the gated penstock intakes.

Canals, flumes and forebays must be designed carefully to minimize hydraulic losses and to ensure that adequate flows reach the power facilities in a hydraulically acceptable manner. Capacity, cross sectional geometry, roughness, and layout must be engineered concurrently to ensure the adequate performance of the conveyance system. With so many variables, channel, flume and forebay arrangement options are virtually unlimited and therefore tend to be project specific. However, the principles remain the same from project to project, as do many of the common issues related to operation and maintenance.

5.2.2 Problems

Inadequate performance of any hydroelectric facility component can usually be attributed to one of three main causes; inadequate original design, change in performance criteria, or deterioration of project components. Common problems encountered with the operation of canals, flumes and forebays can readily be grouped into one of these three areas.

Conveyance systems constructed in accordance with the original design requirements, but which have never operated and/or performed to the levels required by the original design, likely have inherent design flaws. These can range from errors in the assumed roughness of the flume surfaces or canal lining, to errors in the assumed flow patterns within a canal or forebay. Mistakes in the hydrological analysis of the watershed, resulting in different water levels and flow criteria, with the conveyance system are often causes of inadequate performance.

Similar problems can be encountered when the criteria for a given project change during its service life. Changes in project criteria may include increased Probable Maximum Flood (PMF) levels, increased flow requirements due to unit upgrades, changes in seismic criteria, or simply, changes in operating regimes, resulting in conditions not accounted for in the original design, or degradation of conditions, such as rougher water structure surfaces.

Moreover, inadequate original design, or changes in the project criteria, often lead to the failure, and/or significant deterioration, of a particular component of the forebay, flume or canal. For example, overtopping of a canal embankment due to miscalculation of the design flood or the failure of the gate systems controlling flow into the canal, may lead to the failure of an adequately designed embankment. This point cannot be emphasized enough in the evaluation and rehabilitation process.

An example of the catastrophic effects a canal failure can have on the entire project is illustrated by the Swift 2 Power Canal Failure. In that case, the embankment section adjacent to the intakes for the power penstocks failed leading to uncontrolled release of the waters within the power canal (Figure 5.2-1). This uncontrolled release resulted in the inundation of the powerhouse with water and debris, undermining of the foundation, destruction of the facility's electrical equipment, and severe damage to the switchyard.

Canals are also constructed by being excavated from rock, and failure of a canal wall can occur due to leakage and ensuing displacement of the natural stone blocks, as occurred in the unlined rock canal of the Glen Park Hydroelectric Project in Watertown, NY (Figure 5.2-2).

Figure 5.2-1 Canal Failure (courtesy of FERC)

Figure 5.2-2 Glen Park Hydroelectric Project (courtesy of Kleinschmidt Associates)

As stated previously, canals, flumes and forebays are water conveyance structures and therefore must be designed for specific flow requirements. Items such as hydraulic capacity, cross sectional geometry, roughness, and layout are, therefore, fundamental to their successful operation and are affected by the following:

- Erosion or sloughing of embankment slopes resulting in reduced cross sectional area and/or changes in the wetted perimeter and hydraulic roughness.
- Spalling of concrete canal linings, flumes or guide walls as well as corrosion of a steel flume resulting in increased roughness, aquatic growth on walls.
- Sedimentation within the canal, flume or forebay.
- Water loss in the form of seepage through linings, joints or embankments that may impact project performance and endanger the stability of the structures; i.e., foundation failures.
- Plant and animal damage.
- Formation or collection of ice within the canal, flume or forebay.
- Collection of debris on the surface, suspended within the flow, or deposited along the invert of the conveyance system.

These represent only a handful of the problems that can arise but are representative of the types of problems encountered.

5.2.3 Corrective Measures

Solutions to the problems encountered with the operation of canals, flumes and forebays are as diverse as the problems themselves. Nevertheless, common solutions and corrective measures have been established over time in response to the typical problems described above. These solutions are presented below.

- Eroded or sloughing embankment slopes can be replaced in kind via placement of fill material, retained through the installation of a retaining structure, or stabilized by shotcrete applications, or the removal of sloughed materials if the geometric change does not result in stability or safety issues for the embankment.
- Spalled surfaces of flumes, canal linings or guide walls can be rehabilitated through concrete replacement and patching, shotcrete applications, overlays, or similar activities. Corroded surfaces can be cleaned, ground smooth and recoated if the corrosion is not to a degree which requires patching or replacement of the steel.
- Sedimentation within the canal, forebay and to a lesser degree, flume, can be removed via dredging, vacuum extraction, flushing, dewatering and mining of material in the dry; the addition of a stilling basin or similar structure upstream of the canal, flume or forebay to settle out future sediment; or by reducing the cross sectional area of the canal, flume or forebay to increase flow velocities and pass the sediment downstream.
- Seepage through joints, linings or embankments can be remedied through localized grouting of cracks and joints, replacement of joint materials and/or waterstops, installation of a watertight membrane or cutoff wall, or foundation grouting.
- Ice accumulation can be minimized within the canal, flume or forebay through the installation of an ice boom, skimmer wall, or similar structure at the entrance and/or through manipulation of the flow within the conveyance system.
- Debris buildup can be mitigated using a trash boom, or similar techniques employed in avoiding ice within a forebay, flume or canal.

The applicability and relevance of the solutions presented above depend on a multitude of factors, including technical viability, constructability, economics, and schedule. For example, problems caused by sedimentation can be remedied by various means, two of which could include opening downstream gates and 'flushing' the channel, or dredging. Potentially, each will work but will impact operations in very different ways. Flushing would be relatively fast and inexpensive but would require the spilling of a large quantity of water, which has definite value to a hydroelectric facility. In addition, downstream release limitations or the physical limitations of the plant may prohibit the flushing technique, at which point dredging becomes the viable option. However, it too will have to be carefully reviewed from an operational point of view (water levels to support the dredging activities), environmental (disposal of the dredged material), schedule (dredging takes time for mobilization and completion) and many other aspects.

5.2.4 Opportunities

Canal, flume and forebay operation are essential to the conveyance of water and ultimately the production of power for a facility. Regular inspection and maintenance is required to ensure optimal performance and can greatly extend the service life of a project. Regular maintenance may include repairs of cracked or deteriorated concrete in flumes, canal linings and/or forebay structures; inspection and maintenance of the flume structures, including the supports and foundations, maintenance of slopes to ensure flow restrictions due to vegetation and debris build-up are avoided; periodic removal of debris and sediment within the waterways to maintain target flows and velocities; and the installation of floating booms, or other structures to inhibit accumulation of debris or ice within the system.

As structures reach the end of their design service life, larger problems requiring more serious and costly solutions are likely to develop. Moreover, changes in design criteria, philosophy, or regulations may require the upgrade or replacement of system components. When this occurs, the Owner is faced with a myriad of solutions ranging from the economically attractive temporary fix, to the costly, but guaranteed, long term solution. Selection of the proper rehabilitation plan must involve evaluations of safety, risk to the facility and public, constructability issues, environmental aspects, schedule and, of course, economics.

However, additional benefits can often be realized when doing major rehabilitation. These opportunities must not be overlooked or undervalued. For example, if a canal must be dewatered to repair seepage that may result in failure of a slope, the canal can also be cleaned of any accumulated debris and sediment. By taking advantage of the dewatered state, significant savings can be realized in both cost and schedule for the debris removal, and the increased flow may actually boost generation. Another example involves a situation where the canal and forebay have not been sized appropriately for a newly calculated PMF. Fixes could include raising of the embankments, construction of parapet walls, or the introduction of a new spillway. All are feasible, yet a combination of them may actually serve a greater good for the project. By raising the embankments, but incorporating a fuse plug type spillway, overtopping protection is provided for the embankments while providing additional spill capacity to the conveyance system and possibly mitigating the need for costly improvements to downstream structures.

Further opportunities resulting from the need for rehabilitation and upgrade and canal, flume and forebay facilities are presented in the following section.

5.2.5 Case Histories

No. 1 Emergency Breach of Canal Embankment (Flood Condition)
Constantine Project (AEP, 1989)

The Constantine Hydroelectric Project is located in Michigan and consists of a small, four-unit powerhouse. Water is conveyed to the powerhouse via a 1,200-foot long intake channel which runs parallel to the river. Flow into the canal is regulated by a head gate structure which is located at the upstream end of the canal. The canal is formed/contained by earthen embankments, one of which separates the canal from the river, while the landside embankment prevents the canal waters from spilling onto adjacent lands.

During the flood of record, the water level within the channel rose, and was rapidly approaching the crest elevation of the earthen embankments. The water levels in the canal could not be isolated from the rising water in the river, as plant personnel were unable to lower the head gates to stop the inflow. In addition to threatening the stability of the embankments, overtopping of the embankments near the powerhouse would result in undermining the foundation of the structure.

A decision was made to breach the riverside earthen embankment to allow the water to flow back into the river in order to prevent the overtopping of the earthen embankments and undermining of the powerhouse foundation. The breach was located several hundred feet upstream of the powerhouse.

The embankment was subsequently repaired, at which time the owner opted to leave a low spot within the embankment to be a dedicated breach location. In this way, were the gates to become jammed again during a flood event, the embankment would fail, thereby eliminating the potential for powerhouse undermining, or overtopping of the embankment on the opposite side of the channel, causing flooding of downstream houses. A second option was also considered, and involved the construction of an auxiliary spillway within the embankment. However, since the project has a generation capacity of only 1.2-MW, the economics did not justify the construction of an auxiliary spillway.

No. 2 Ice Induced Failure of Forebay Skimmer Wall
Safe Harbor Project (MWH, 1999)

The Safe Harbor Hydroelectric Project is located on the Susquehanna River in Pennsylvania. The run-of-river project consists of a large, reinforced concrete dam and gated spillway across the river with a surface-type, 412-MW powerplant constructed on the left bank. The powerplant is separated from the river by a 1,500-ft long skimmer wall. The skimmer wall, originally constructed in the 1930's with the rest of the project, consisted of reinforced concrete piers which stood approximately 67-ft above the river bed and supported a 22-ft high section of wall at the water surface. The purpose of the skimmer wall was to prevent the inflow of debris and ice into the powerhouse forebay.

On January 20, 1996, a natural ice dam that had formed across the Susquehanna River upstream of the project broke, instantaneously releasing a large amount of water and ice towards the Safe Harbor project. The river area adjacent to the project already had significant amounts of surface and suspended ice, a portion of which was lodged against, and under, the existing skimmer wall. Water was being released over the spillway in an attempt to dislodge and pass the accumulated ice. In addition, water was being released through the powerhouse at near full generating capacity.

It is believed that the ice build-up adjacent to the skimmer wall had reached a level which significantly restricted inflow into the forebay. With the plant generating at full capacity, water was being withdrawn from the forebay at the maximum rate. With the arrival of the surge wave from the failure of the upstream ice dam, a water level differential developed between the river and the forebay. The loads on the wall were too great and approximately two thirds of the existing 1,500-ft long skimmer wall collapsed. The failure of the wall resulted in a large, uncontrolled inflow of water and ice into the forebay that overtopped the intake deck of the powerhouse and ultimately required each of the generating units to be temporarily shutdown.

The one-third portion of the wall that remained was the section closest to the spillway and powerhouse. This portion was also damaged in that the top of the structure shifted approximately six inches from its original position. An underwater inspection at the base of the remaining wall revealed that the anchorage at the top of the footing had begun to fail which likely accounted for the measured displacement.

Rehabilitation and replacement options were investigated, ranging from complete demolition and replacement of the structure in-the-dry, through the installation of a substantial cofferdam system, to a multitude of in-the-wet solutions. At the same time, a review of the original structure was performed which included a review of the original design criteria, and a condition assessment of the existing structure in terms its functional use to the Owner.

Ultimately, an innovative, in-the-wet option (tremie-concreted), and post-tensioned piers was selected and implemented (Figure 5.2-4). The new skimmer wall was designed to meet current standards for seismic, ice and other known loading conditions. The new wall incorporated a fuse-plug design, in that the skimmer wall was constructed of vertically stacked, sacrificial stoplogs, designed to fail under extreme loadings prior to the failure of the piers supporting the stoplogs. In this way, replacement of the stoplogs is all that would be required should an extreme event occur again in the future. In addition to being designed for current codes and design criteria, a fish passage system was incorporated into the design of the replacement skimmer wall. The remaining portions of the original skimmer wall were abandoned in place.

By December 1999, the replacement skimmer wall was successfully constructed in-the-wet, saving tens of millions of dollars in cost and months in construction duration, versus traditional in-the-dry methods utilizing cofferdams and staged construction.

No. 3 Deterioration of Water Conveyance System Naches Project (deRubertis, 1996)

The Naches water conveyance system consists of both lined and unlined, trapezoidal shaped reaches of canal, three elevated, rectangular concrete flumes and one, partially lined horseshoe-shaped tunnel. The Wapatox canal is the principal feature of the Naches water conveyance system and draws an average annual flow of approximately 400 cfs from the Naches River, serving the Drop and Naches powerhouses, each with integrated upstream forebays. Other features of the system include a waterway and two spillways.

Due to the advancing age of the facility, construction was completed in 1909, a study was performed in 1995 to assess the condition of the system, identify methods and costs of repairing or replacing conveyance structures based on a least cost method for a 30-year life, a maximum allotted annual expenditure, and risk of failure.

The canal and water conveyance system was found to be in reasonably good condition due in large part to the annual maintenance program instituted at the project. The program included activities which repaired obvious local defects in the conveyance structures, as well as larger projects performed over the years, including the complete replacement of lining sections within various sections of the canal. However, a study did identify several sections of the canal embankments which were susceptible to failure in their current condition. In addition, portions of the concrete lining within the canal were found to be damaged due to root growth, and several areas of the concrete lining exhibited severe deterioration. The flumes were also found to be deteriorated as were the spill walls in both forebays.

Following the field inspection, alternatives for repair of the canals and flumes were studied, design sketches were prepared, quantities were estimated, and contractors and suppliers were contacted to establish budgetary level pricing quotes. Based on this data, repairs were prioritized and scheduled based on a need/risk of failure and relative costs, so as that the allotted annual maintenance expenditures were not exceeded.

It was determined that the flumes posed the highest risk of failure and were slated for replacement with conventionally cast concrete elevated flumes in sequential years based on their relative conditions. Elevated steel flumes offered comparable construction costs but required more maintenance, while steel pipe siphons would cost more but would take less time to construct. Other scheduled repairs included replacement of forebay spill walls and canal lining repairs. The repairs to the flumes, forebay spill walls, and canal linings were implemented in the time period 1996-2000 within budget. The conveyance system has worked well since completion of the repairs.

No. 4 Upgrade of Forebay and Pipeline
Santa Ana River Hydroelectric Project (MWH, 2001)

Southern California Edison's Eastern Hydroelectric facilities, located in the Santa Ana River Canyon, included the 0.8-MW Santa Ana River No. 2 (SAR-2) powerhouse, the SAR No. 3 water conveyance system (SAR-3 flow line), and the 1.2-MW Santa Ana River No. 3 (SAR-3) powerhouse.

Approximately 83-cfs of water was delivered from the SAR-2 forebay to the SAR-2 powerhouse via a 36-inch diameter steel pipe. The 12,300-foot long SAR-3 flow line conveyed flows up to 88-cfs from the SAR-2 tailrace to the SAR-3 forebay across the Northern San Andreas Fault via a concrete flume, Lennon type flumes, and rock tunnels. At the SAR-3 forebay, water was delivered to various municipal water agencies as well as to the 40-inch diameter SAR-3 penstock. The SAR-3 powerhouse, at an elevation 686-ft below that of the SAR-2 forebay, generated power using the pressured water from the penstock and returned it to the river through a concrete lined channel.

The United State Army Corps of Engineers (USACE) constructed the Seven Oaks Dam, a flood control project with storage capabilities, near the mouth of the Santa Ana River Canyon. As a result of this dam, water levels in the upstream reservoir area could rise as a result of normal flood control operations and inundate the SAR-2 powerhouse and SAR-3 flow line. In addition, the tunnel forming a portion of the SAR-3 flow line passes through what is now the left abutment of the Seven Oaks Dam.

To mitigate this situation, the project was rehabilitated to include a new closed flow line (penstock) from the existing SAR-2 forebay structure to a new 3.11-MW SAR-3 powerhouse. The existing SAR-2 powerhouse was abandoned in place, with all electrical and mechanical equipment removed. A new headbreaker valve building was incorporated into the flow line to provide continued water supply to the water agencies at the SAR-3 forebay.

The new flow line includes a 40-inch diameter buried steel pipe constructed from the SAR-2 forebay to the left abutment tunnel, a portion of which is designed as an inverted siphon. The pipeline surfaces briefly before entering the tunnel. The annular space around the pipe within the tunnel is sealed with a reinforced concrete plug to prevent the potential escape of water from an impounded reservoir.

Another active fault was identified by the Corps which crosses nearly perpendicular to the tunnel near its midpoint. As such, a protective oversized culvert was placed around the pipeline to allow for minor displacement along the fault without direct impact to the pipeline itself.

At the downstream portal, the pipeline remains exposed for approximately 200-ft until it enters the new headbreaker valve building. Within this building is housed an emergency control valve which is to be used to maintain a pressurized pipe in the inverted siphon reach, in the event that the penstock is breached downstream of the building. Immediately downstream of the emergency valve is a tee fitting connected to the SAR-3 penstock. The high-branch line is connected to a head-breaking polyjet valve which discharges into the existing rock tunnel that feeds the old SAR-3 forebay, from which the water agencies obtain their supply.

A new SAR-3 powerhouse was constructed to replace the previous SAR-2 and SAR-3 powerhouses. The new, single unit powerhouse is located on the same building footprint as the previous SAR-3 powerhouse and discharges a maximum flow of 90-cfs into the existing concrete lined tailrace tunnel.

No. 5 Plant Expansion and New Forebay Dam
Grand Coulee Hydroelectric Project (USBR, 1981)

When the Grand Coulee Project was originally constructed, it included provisions for the addition of third powerplant. The addition of this third powerplant however required the extension of the main dam, the extension being aptly labeled the Forebay Dam for the Third Powerplant (Figure 5.2-3).

Figure 5.2-3 Grand Coulee Forebay Dam and Third Powerplant (courtesy of USBR website)

The Forebay Dam, though actually an extension of the main Grand Coulee Dam, is a gravity structure over 200 feet high and more than 1.300 feet long that is constructed on a rock bench 270 feet above the downstream powerhouse. Placement of the dam atop this bench required extensive testing and studies to determine the stability of the foundation in which were included two major faults and five major joint sets.

Numerous analyses and studies were performed to determine whether or not the placement of the new dam atop the rock bench above the new powerhouse was acceptable. These included exploratory drilling programs and laboratory studies to determine the parameters of the rock foundation. These were used in analyses, including determination of the sliding stability of the dam along the downstream sloping joints; rigid block analyses of possible rock blocks formed by the existing faults and joint sets; and finite element studies of the dam foundation and backslope to determine stability and the required foundation treatment.

The studies were completed and the results were found to be complete and adequate. This allowed the construction of the present day Third Powerplant and the associated Forebay Dam atop the steeply sloped rock bench, high above the powerhouse.

5.2.6 Collective Knowledge

Canals, flumes and forebays are an integral part of hydroelectric generation and have been around as long as any hydroelectric plant. As such, substantial amounts of literature regarding the design, maintenance, protection, and inspection of these facilities have been produced and much of the information presented is still valid and quite useful. Some of the more well-known and complete references include:

1. Linsley, Ray K. & Franzini, Joseph B. (1972). 2^{nd} Edition. *Water Resources Engineering,* McGraw-Hill Book Company, New York, NY.
2. United States Army Corps of Engineers (USACE). (n.d.). *Engineering Manual, Civil Works Construction, Part 114.* USACE, Washington, DC.
3. United States Bureau of Reclamation (USBR). (1987). *Design of Small Dams.*, US Bureau of Reclamation, Washington, DC. [Water Resources Technical Publication].
4. United States Bureau of Reclamation (USBR). (1974). *Design of Small Canal Structures,* US Bureau of Reclamation, Denver, CO. [USBR Technical Report].
5. United States Bureau of Reclamation (USBR). (1967). *Design Standard 3: Canals and Related Structures*, USBR, Denver, CO. [Design Standards].
6. United States Bureau of Reclamation (USBR). (1963). *Linings for Irrigation Canals.* USBR, Denver, CO. [USBR Technical Report].
7. Zipparro, V. & Hasen, H. (1993). 4^{th} Edition. *Davis' Handbook on Hydraulics.* McGraw-Hill Book Company, New York, NY.

5.2.7 Technical References

American Electric Power (AEP). (1989). Project files for the American Electric Power, Columbus OH.

Davis. (1993). *Davis' Handbook on Hydraulics*, 4th Edition, edited by Zipparo, V. and Hasen, H., McGraw-Hill Inc., New York, NY.

deRubertis. (1996). Personal files, 1995-1996.

Doland, James J. (1954). *Hydro Power Engineering*. The Ronald Press Company, New York, NY.

MWH. (2001). Project files for Safe Harbor Water Power Corporation. MWH, Chicago, IL.

MWH. (2001). Project files for Southern California Edison, CA. MWH, Chicago, IL.

United States Bureau of Reclamation (USBR). (1981). http://www.usbr.gov/power/data/sites/grandcou/grandcou.html.

5.3 Tunnels, Shafts, and Other Underground Openings

5.3.1 Function

Tunnels, shafts, and other underground openings come in a variety of sizes, shapes, configurations, and designs. Most underground openings for hydroelectric schemes are made in rock. Ground support, as required, is selected from a range of devices and techniques such as rockbolts, shotcrete, weld mesh, straps, steel sets, and timber cribbing. Conventional mining (drill and shoot) methods produce rough rock surfaces. Tunnel boring machines produce smooth rock surfaces. Some are lined with concrete, steel, or a combination of both concrete and steel. Some are unlined. Many are partially lined, especially where ground pressures are less than water pressures in the tunnel. Rock traps to capture sediment or rock are found in some tunnels. The traps reduce turbine metal wear by preventing suspended sediment and rocks from becoming entrained in the turbine supply flow.

Tunnels and shafts are common features of conveyance systems carrying water for power between two points. Underground powerhouses often complement hydroelectric schemes. Load rejection and acceptance may be accommodated in several ways. One common arrangement is to convey water for power in a tunnel at low head, followed by a transition to a steep penstock to the powerhouse, with surge handled in a surge tank at the transition. Another common arrangement is for the tunnel (or tunnel and shaft) to accept surge without benefit of any other special provision. Occasionally, an enlarged underground opening is provided for surge protection or to provide a chamber for a gate.

5.3.2 Problems

The most common problems are operational instability, leakage, and differing site conditions. These problems may affect all underground openings including tunnels, shafts, caverns, and adits used for access or drainage. Instability may take the form of collapse or loss of ground due to operational conditions. Collapse may completely block an underground opening, whereas loss of ground may partially block an underground opening. Faults intersecting tunnels are often weak zones requiring special treatment. Hydraulic jacking, resulting from unbalanced pressures, may invite large-scale instability in the rock mass containing the underground opening. Leakage from an underground opening may contribute to instability at the ground surface by increasing the pore pressures in rock or soil, or by transporting erodable rock or soil. Leakage may also constitute an unacceptable economic loss of water. Inadequate investigation or evaluation may result in site conditions that differ from those estimated for design.

Typical causes of tunnel problems are as follows:

- Instability leading to collapse is caused by the underground opening's inability to sustain the loads placed on it. Stability of an underground opening relies on the ability of the rock mass around the opening to redistribute the stresses acting on it. Fortunately, most serious problems of instability manifest themselves during construction, allowing for appropriate reinforcement to be applied.
- Loss of ground often occurs when flow exploits discontinuities in the rock mass, allowing blocks to become detached and fall. Simply put, this is erosion, a common phenomenon in unlined underground openings. Gravity, acting over a number of years, may also cause key block to become detached and fall.
- In general, earthquakes do little, or no damage to underground openings, unless ground rupture occurs within the opening. Many underground powerhouses have survived strong ground shaking without significant damage.
- Tunnels in moving ground (active landslides) experience stresses that may greatly exceed those in stable ground. Tunnels near a valley wall and oriented roughly parallel with the valley may require reinforcement to account for low minor principal stresses.
- Leakage occurs from an underground opening when the confining stresses in the rock mass are less than the hydraulic stress, or when a privileged path of leakage exists. An important concept is to accept that concrete linings will crack, whether reinforced or not, and the cracks will provide a path for leakage from an underground opening no matter how carefully the concrete work is done. Steel lining is often employed to preclude leakage.
- Failure to properly investigate and evaluate how the ground will behave during tunneling and operation.

5.3.3 Corrective Measures

Reinforcement of the ground is the most common solution for instability, and reinforcement may take several forms:

- Lining with steel pipe with concrete backfill in the annular space between the rock and pipe, and with contact grouting behind both.
- Concrete lining, either plain or reinforced, either grouted or ungrouted.
- Steel sets (ribs made of wide flange beams).
- Rock bolts and straps.
- Shotcrete.
- Timber cribbing.

Combinations of reinforcement are common. For example, steel sets may be employed with blocking and lagging, rock bolts, and shotcrete. Reinforcement can be effective, either for instability leading to collapse, or for simple loss of ground.

Grouting and drainage methods are often employed to control water pressure surrounding underground openings.

In general, hydraulic jacking of the rock mass is avoided by extending steel lining to a depth in rock cover of at least one half the static hydraulic head.

Three simple steps can be effective in choosing the appropriate corrective action:

1. Know the ground and know the stresses applied to the opening by the rock and the water. Knowing the ground implies knowledge of the quality of the rock mass, its discontinuities, groundwater conditions, rock materials and jointing patterns. The terrain, coupled with rock properties, will allow an estimate of the stresses in the rock mass. Understand the minor and maximum principal stresses as well as the vertical stress. Use the hydraulic grade line to provide an estimate of the hydraulic stress. Thorough investigation will reduce the risk of encountering differing site conditions.
2. Know the design of the underground opening. Understand what reinforcement exists, how it was applied, and its condition.
3. Perform an inspection. Inspection is required to know the condition of the tunnel, shaft, or other underground opening. Care and judgment should be exercised in planning for, and executing, dewatering of any underground project feature that functions at substantial pressure, especially if it is unlined. Too rapid dewatering may permit unbalanced stresses to develop and lead to local or general failure of the rock mass. A recent trend is to inspect remotely without dewatering, using an underwater remotely operated vehicle (ROV). An ROV may reduce inspection time and save the cost of dewatering and rewatering; but it may not reveal the same level of detail as a dewatered physical inspection.

5.3.4 Opportunities

The most common opportunity for tunnels, shafts, and other underground openings is to reduce or avoid trouble, thereby reducing maintenance costs. There may be possibilities for reducing friction losses as well. For example, an unlined (rough) tunnel might be lined (smooth) if the costs to line are recovered from the savings resulting from reduced friction.

5.3.5 Case Histories

No. 1 Structural Instability of Tunnel
Terror Lake Project (de Rubertis, 2001)

The Terror Lake Hydroelectric Project, in a remote area in Kodiak Island, Alaska, was completed in 1983 and is owned by the Four Dam Pool Power Agency. Water for power is conveyed from a reservoir through a 5-mile-long, 11-ft-diameter, machine-bored tunnel. Flow from a creek is added to flow in the tunnel through a 625-ft-long, 45° inclined, machine-bored shaft. The hydraulic grade line passes through the shaft about 200 ft below the entrance to the shaft. The main power tunnel is designed to carry 435 cfs with 1,322 ft of head.

Inspection of the tunnel revealed a large mass of broken rock lying at the base of the shaft. The discovery was made when the tunnel was dewatered for a routine 5-year safety inspection.

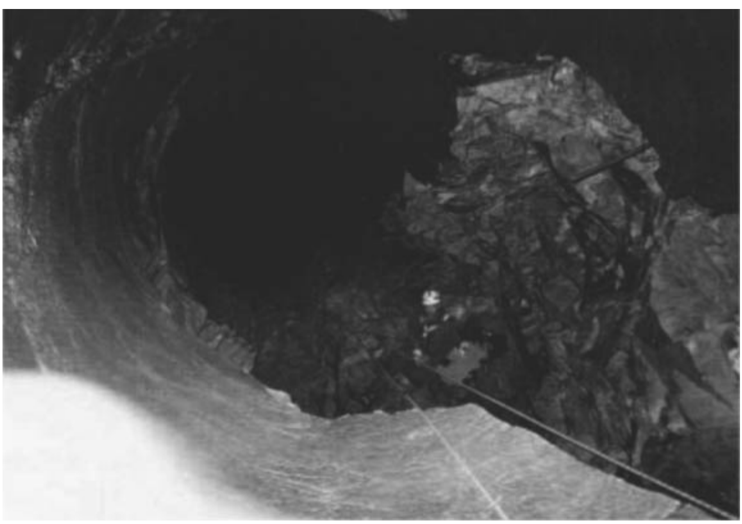


Figure 5.3-1 Terror Lake – Damaged Tunnel (courtesy of de Rubertis)

Detailed inspection, geologic mapping, and measurements of the source of the broken rock in the shaft found that perhaps as much as 150 cy of rock was lost from a weak rock zone at a point in the shaft where flow in the shaft reached the hydraulic grade line (or free water surface). Reinforcement during construction consisted of a few rock bolts and a thin application of shotcrete in the weak rock zone.

The rockfall in the shaft occurred because weak rock was present at the point in the shaft where the plunging water in the shaft struck the free water surface. As the energy of the plunging water was dissipated, it was able to perform work on the rock mass, causing it to fail. This problem was not foreseen during design and construction, and inadequate reinforcement was applied.

Four alternatives were studied:

- No action. Allow the water to continue to perform work on the rock mass until stability is reached.
- Reinforce the rockfall area with rockbolts and shotcrete, keeping the rockfall dimensions roughly unchanged.
- Place a steel liner through the rockfall area, and backfill behind the liner with concrete.
- Place a steel liner for the full 625-ft length of the shaft, and backfill behind the liner with concrete.

Steel lining was installed for the full length of the shaft, and the annular space between the lining and the rock mass was backfilled with concrete. Given the remote site of the project, this solution to the problem was selected to preclude the possibility of needing to perform any future work in the shaft.

The benefits to the owner were reduced short-term and long-term maintenance costs.

No. 2 Leakage of Concrete Tunnel Liner
Packwood Project (Passage, 2002)

At Energy Northwest's Packwood Hydroelectric Project, water for power is conveyed through a series of tunnels and pipelines. Tunnel 2 was completed in 1964 and is 3,200 ft long, horseshoe-shaped, conventionally mined, and about 9 ft high. At the downstream end of the tunnel is a transition to concrete cylinder pipeline. The rock mass that contains the tunnel is mixed ground, predominantly hard, widely jointed felsite with several interbedded zones of weak volcanic sediments (shale and sandstone). The hillside containing Tunnel 2 is a large, active landslide block exhibiting evidence of continued movement. The entire invert of the tunnel is lined with unreinforced concrete. Sections of the tunnel in weak rock are fully lined with plain concrete. The tunnel is about 75 ft below the hydraulic grade line and conveys approximately 200 cfs.

Periodic inspections during annual maintenance outages revealed that the concrete lining was cracked and the invert slab heaved in several of the lined tunnel sections. Leakage of several cfs at the ground surface near the transition to pipeline was evident. On at least two occasions, the leakage saturated the hillside soils, causing landslides below the tunnel. Most of the leakage from the tunnel was judged to originate in the downstream end, in a section roughly 300 ft long.

Cracking of the liner was believed to result from thin-bedded shale, sandstone, and tuff too weak to resist the pressures being applied to the unreinforced concrete lining. Attempts to reduce leakage were frustrating. Several generations of rigid patching materials (cement-based products) failed to reduce leakage. Offsets in the patches confirmed continued distress. Flexible patching materials such as joint sealants, oakum, and lead wool also proved ineffective. Chemical grouting was attempted without success. Steel bracing and rockbolts were applied to one lined section and may have reduced the rate of cracking; however, continued heaving of the invert required periodic replacement.

Alternatives studied included installing a pipe, cement grouting, and applying reinforced shotcrete. Finally, a decision was made to take an approach that combined both structural and flexible elements to reduce or eliminate the leakage. The following actions were taken to:

- Clean and smooth the tunnel perimeter by feathering out cracks and covering depressions in the lining.
- Apply 45-mil thick PVC non-woven membrane to the tunnel perimeter.
- Erect 4-in steel sets slightly smaller than the tunnel size, with steel channels between the flanges of the sets that were spaced on 4-ft centers.
- Cast a new 9-in thick invert slab.
- Backfill the walls behind the steel channels with concrete.
- Grout any voids remaining in the crown with neat cement grout.
- Create a leak-proof transition to the pipeline, and relocate power and control cables in new cable trays.

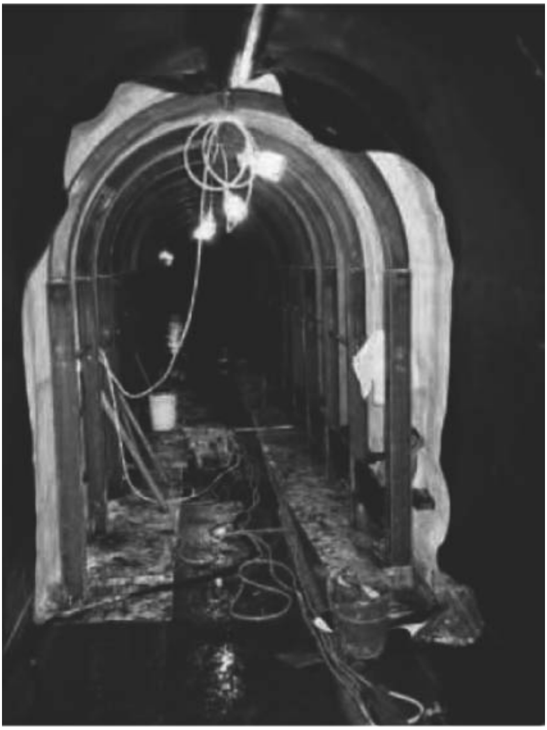


Figure 5.3-2 Packwood – Flexible Liner (courtesy of de Rubertis)

During repairs, it was discovered that the original concrete lining in the repaired section was placed against steel sets and timber lagging and not directly against the excavated rock line. This construction technique is likely to have contributed to the cracking of the concrete lining and provided a path for water flow behind the lining.

The corrective action eliminated leakage. The benefits to the owner were reduced maintenance costs and regulatory compliance.

No. 3 Inadequate Investigation of Subsurface Conditions Power Creek Project (de Rubertis, 2002)

The Cordova Electric Cooperative owns the Power Creek Hydroelectric Project, completed in 2001. The project is located near the town of Cordova, Alaska. Water is conveyed from the intake to the powerhouse in a penstock, a portion of which passes through a 3,000-ft-long tunnel. Within the tunnel, the penstock experiences 200 ft of head. The conventionally mined tunnel is nominally 9½-ft-high and 9 ft wide. The rock mass containing the tunnel is closely jointed and faulted ground, consisting of highly metamorphosed argillite and greywacke, along with sub aqueous volcanic rock (greenstone).

The project was constructed turnkey. The tunnel was planned to be an unlined pressure tunnel with appropriate steel lining at the downstream and upstream ends. During the design phase, a single boring was attempted near the downstream end of the tunnel alignment but could not be completed because of bad ground.

As mining progressed, numerous expedient (split set) rockbolts were needed for initial support to stabilize the ground to allow mining to proceed. Conditions were exacerbated by groundwater inflows of several thousand gpm. Mining was completed, with difficulty, and alternatives for a permanent support solution were studied. Any type of concrete lining was dismissed because of high groundwater inflow.

Investigation and evaluation of the ground during the design phase were inadequate. Differing conditions led to abandonment of the design for an unlined pressure tunnel. As a consequence, the project suffered delay and high extra cost.

No feasible alternative to extending the penstock through the tunnel could be identified. The permanent support solution selected consisted of:

- Placing heavy steel sets with timber lagging between the flanges and backfill concrete behind the lagging in the worst ground.
- Installing galvanized, fully resin-anchored, pattern rockbolts (3 different patterns depending on the condition of the ground).
- Installing galvanized mine straps across weak areas.
- Installing weld wire fabric between rockbolts in raveling ground.
- Installing timber cribbing to support rock haunches that could not be effectively rockbolted.
- Shotcreting the entire back and ribs of the tunnel in all areas of permanent support. Shotcrete covered at least 75% of the tunnel area. A few tunnel sections required no permanent support.

Figure 5.3-3 Power Creek (courtesy of de Rubertis)

After completing the permanent support, a 76-in diameter steel penstock was installed inside the tunnel. In areas of weak ground, the top of the steel penstock was covered with a protective layer of timber.

The owner did not benefit from the differing site conditions. Rather, the owner suffered delay and extra cost.

No. 4 Fault Zones and Deteriorated Concrete Liner Spirit Lake Project (USACE, 1998)

The Spirit Lake Outlet Tunnel was constructed in 1984-85 to provide an outlet to Spirit Lake following the eruption and blockage of the former outlet by a landslide that preceded the eruption of Mount St. Helens on 18 May 1980. The 8,484-foot long 11-foot diameter tunnel was excavated through rock consisting of primarily volcanic ejecta (tuff) and flow rocks (primarily andesite). Of the total tunnel length, 8,220 ft was excavated utilizing a tunnel-boring machine (TBM) and 264 ft was excavated by drill and blast. During the mining of the tunnel, several fault zones were encountered which required the installation of steel supports (w 4 x 13). The worst ground conditions occurred where faulting intersected the tunnel between Sta 73+32 and Sta 73+80 and between Sta 74+90 and Sta 75+56, over a mile from the outlet portal and access point at Sta 15+20. These areas were excavated with a great deal of difficulty, and the heavily sheared and squeezing ground condition required modification to the contractor's original support design. The modifications included the installation of jump sets, and the removal and replacement of the precast invert segments with steel invert struts encased in cast-in-place concrete. Inspections of the completed work in 1987 and 1988 indicated deterioration of the shotcrete lining at the contact between the arch shotcrete and the cast-in-place invert concrete, which is also the normal water flow line of the tunnel. Concrete repair was performed in October 1988 between Sta 73+50 and Sta 74+60. Subsequent inspections in October and December 1992 showed that deterioration of the same areas had occurred again, including significant buckling and shearing of the steel supports. This area was repaired with hand-placed concrete in January and February of 1993.

During the concrete repair completed in 1993 the exposed ribs were mapped. Most ribs, right and left, were found to be sheared (some completely), cracked, and/or deformed. Most of the damage to the ribs and concrete occurred below the springline and appeared to coincide with water levels typical in the tunnel. Material samples of the tunnel wall were collected and analyzed in 1992 and 1993. Studies were conducted to assess the specific cause or causes leading to the failure of the ribsets and tunnel lining. No conclusive single cause was identified, but a number of factors were identified that may have contributed to the tunnel concrete/ribset system failure. These factors included a continued squeezing to the tunnel in this shear zone; erosion behind the ribs, decreasing the contact area between the ribs and rock, and allowing the use of smaller ribsets not consistent with the original design; an inherent weakness in the rib/invert segment connection design; corrosion and the resultant reduced capacity of the ribs; and the possibility of a reaction taking place between the ribs and steel fibers in the concrete and/or ions in the clay material surrounding the ribs.

In 1996, a new repair design was finalized for the repair zone from Sta 73+48 to 74+52. The contract required the contractor to mechanically excavate the concrete between the existing ribsets on 2 ft or 4 ft centers without removing or harming those ribsets, install temporary ground support, and new steel ribsets (w 8 x 28), and place new concrete and shotcrete. The contractor utilized a "roadheader" to excavate the slot between the existing ribsets and, using a special attachment, lift and placed the 120-degree ribset segments into position for blocking and bolting. A hand-packed concrete grout was used for blocking. 34 ribsets were installed. The final actions included placing concrete in accordance with the contract up to the springline and spraying a fine coat (1'" to 2") of shotcrete over the remaining exposed ribs above the springline. At this time, Spirit Lake was filling at a rate that approached its limit, El 3460, and would require the tunnel to be opened to pass flow and stabilize the lake level. The Contractor was required to demobilize from the tunnel and the contract closed out. The remaining concrete to be placed above the springline was installed in the next season.

The benefit to the owner was continued, successful operation of the tunnel.

No. 5 Deteriorated Wood Pipeline (Replacement) Trenton Project (Brookfield Power New York, 1982)

The Trenton Hydroelectric Project was originally built in 1901. The project develops power from the West Canada Creek in central New York. Before remedial action was taken, water was diverted at an intake and conveyed in 7-ft and 12-ft diameter wood stave pipelines, approximately 3,600 ft long. The pipelines had reached the end of their service life.

The original wood stave pipelines leaked excessively and required high maintenance. In examining alternatives to repair or replace the pipelines, a redevelopment opportunity was identified to improve conveyance efficiency in addition to extending the project's service life.

Several replacement alternatives were considered. They included 1) replacement of the pipelines with either wood stave, concrete, fiberglass or steel pipelines following the same route as the existing pipelines, or 2) following a modified route which required construction of a tunnel and surface steel pipeline. The combination of tunnel and surface steel pipeline was chosen because of its lower overall construction cost. The tunnel was concrete lined with a diameter of 14 feet, replacing a 1,265 ft long section of the two original pipelines. The tunnel was mined conventionally in "moderately blocky and seamy and very firm ground" in the horizontally bedded Trenton Limestone Formation of Ordovician age. Minor overbreak and seepage were experienced. Resin encapsulated rock bolts were installed in a regular pattern throughout the tunnel back. Curtain and contact grouting were performed at required locations. Detailed geologic mapping of the entire tunnel was performed.

The benefit to the owner was the ability to continue operating the 28-MW facility more efficiently, and with reduced operation and maintenance costs. The existing wood stave pipelines were over 70 years old and well beyond their useful life. Leakage was excessive and failure of the pipelines was likely. Replacement of the pipelines with one larger diameter conduit reduced head losses, thereby increasing generation and reliability. Concurrent construction of the new intake structure and surge tank was undertaken, taking advantage of the plant shut down necessary for pipeline replacement work.

No. 6 Tunnel Surge
Roberts Tunnel Hydro (Denver Water, 1995)

A twenty-seven mile long tunnel was retrofitted with a 6 MW Francis turbine/generator. Operating head is 340 feet at 300 cfs. The unit was installed in 1985. The tunnel has two ventilation shafts, one at about the midpoint of the tunnel and a second one about 1,000 feet upstream of the turbine/generator unit.

During installation and initial testing, the unit would not operate at maximum design output without excessive vibrations and a surging problem that was not well understood. The project was commissioned. Standard operating instructions were to operate at, or near 5 MW. After several years of operation, the generator was overhauled because of excessive wear on the horizontal thrust bearing. The symptoms were that the unit would "surge or hammer" forward of the turbine/generator at regular intervals (about 5 seconds apart). This surging ultimately caused the failure of the thrust bearing.

After extensive analysis, it was determined that the unit was not properly designed for the normal head and flow and was operating outside the design range. In addition to this problem, the vent shaft located 1,000 feet upstream was found to synchronize (harmonically) with the unit, effectively doubling the forces (thrusts) on the unit.

Small "turn key" units like this one require the same level of care and design level as larger units. Owners, consultants and suppliers did not evaluate the existing conditions correctly.

It was not considered to be cost effective to replace the turbine or generator, so it is operated at 5 MW. The thrust bearing mounting bracket, bearing, and other structural units were reinforced or replaced. The benefit to the owner was reduced maintenance costs.

5.3.6 Collective Knowledge

1. American Society of Civil Engineers (ASCE). Technical Committee on Tunnel Lining Design. (1984). *Guidelines for Tunnel Lining Design.* ASCE, New York, NY.
2. American Society of Civil Engineers (ASCE). (1971). "Underground Rock Chambers" in *Proceedings of a symposium held during the ASCE National Meeting on Water Resources Engineering, Phoenix, Arizona, January 13-14, 1971.* ASCE, New York, NY.
3. Bickel, J.O. & Kuesel, T.R. (1982). *Tunnel Engineering Handbook.* Von Nostrand Reinhold, New York, NY.
4. Broch, Einar & Lysne, Dagfin K. (1987). "Underground Power Plants" in *Proceedings of the International Conference on Hydropower, Oslo, Norway, June 1987.* International Centre for Hydropower, Trondheim, Norway.
5. Hoek, E. & Brown, E.T. (1980). *Underground Excavations in Rock*, The Institution of Mining and Metallurgy, London.
6. Hunt, R.E. (1984). *Geotechnical Engineering Investigation Manual*, McGraw Hill, New York, NY.
7. Jacobs, John M. & Hendricks, Robert S. (eds.) *(1987) RETC Proceedings: Rapid Excavation and Tunneling Conference, New Orleans, June 1987.* Society for Mining Metallurgy and Exploration, Littleton, CO.
8. Proctor, R.V. & White, T.L. (1946). *Rock Tunneling with Steel Supports.* Commercial Shearing and Stamping, Youngstown, AZ.
9. Stillborg, B. (1986). *Professional Users' Handbook for Rock Bolting.* Trans Tech Publications, Zurich.

5.3.7 Technical References

Brookfield Power New York. (1982) Project files, Brookfield Power New York, Liverpool, NY.

de Rubertis, Kim. (2002). Personal files, 1998-2002.

de Rubertis, Kim. (2001). Personal files, 1999-2001.

Denver Water. (1995). Project files, Denver Water Board, Denver, CO.

Passage, J. *et al.* (2002). "Tunnel Repairs Through a Manhole" in *ASDSO 18th Annual Conference, September 8-11, 2002, Tampa, FL.* Association of State Dam Safety Officials, Lexington, KY.

United States Army Corps of Engineers (USACE). (1998). Project files, USACE, Portland District, Portland, OR.

United States Army Corps of Engineers (USACE). (1997). *Engineering and Design - Tunnels and Shafts in Rock.* [EM-1110-2-2901]. United States Army Corp of Engineers, Hyattsville, MD.

http://www.usace.army.mil/publications/eng-manuals/em1110-2-2901/toc.htm

5.4 Penstocks

5.4.1 Function

Penstocks are pressurized conduits that transport water from the first free water surface to a turbine. At low heads, the penstock may be as short as a conduit from the intake to the turbine spiral case. Characteristics of functional penstocks are structural stability, minimal water leakage, and maximum hydraulic performance. Some of the components of a penstock system include the following:

- Penstock shell (steel, pre-cast or cast-in-place concrete, wood, plastic, or fiberglass).
- Protective system (coating and linings or cathodic protection).
- Supports (concrete, masonry, wood, or steel saddles; ring girders; piers; anchor blocks; or rockers).
- Joints (bellows, expansion, bolted sleeve-type coupling, riveted, or welded).
- Rings (stiffener, thrust, or seepage).
- Penetrations (air valves, vents, drains, piezometer and flow meter taps, and manholes).

Another integral component of the penstock system is a surge tank whose purpose is to help provide plant stability and minimize water hammer. Surge tanks are covered in the tunnel section.

Penstocks, coatings and linings have been the subject of several ASCE Task Committee efforts. These references are listed in Section 5.4.6. The following is simply an overview of issues with aging penstocks and the reader is directed to more detailed resources for further information.

5.4.2 Problems

Penstock shells, whether constructed of steel, concrete, wood, plastic, or fiberglass, are subject to the following issues:

a) Life Expectancy.
b) Protection – Coating and Lining Systems.
c) Wall Thinning.
d) Supports.
e) Joints.
f) Alignment.
g) Loss of Roundness.
h) Dewatering/Watering.
i) Geotechnical.
j) Vandalism.

a) Life Expectancy

The life expectancy of the penstock is normally estimated during the design phase but will vary somewhat from the factors listed below. An inspection and maintenance schedule is established after startup to meet this life expectancy. With proper maintenance, conservative life expectancies for various materials are as follows: steel – 75 to 90 years; wood – 35 to 50 years; concrete – 35 to 75 years; and plastic or fiberglass (depending on treatment for ultraviolet exposure) – 30 to 50 years.

b) Protection – Coating and Lining Systems

Protection systems, i.e. coatings (exterior of penstocks) and linings (interior of penstocks), are critical to the performance and longevity of the penstock. Breakdown and deterioration of the protection system can lead to significant problems. However, a regular program of inspection, condition assessment, and repair/maintenance can prove to be a cost-effective solution to corrosion and other problems. Excellent resources on coatings and linings are contained in EPRI's report on penstock rehabilitation (EPRI, 2000), and other information from the National Association of Corrosion Engineers (NACE) and the Society for Protective Coatings Committee.

c) Wall thinning

Reduction in shell thickness can occur if coatings and linings are not maintained, or as a result of corrosion, erosion or cavitation. Corrosion can occur on both the inside and outside of the shell. If the coating and lining protection system breaks down, deterioration can occur. Wood staves are also subject to deterioration, splintering, and dehydration causing shrinkage, leakage, and possible loss of structural integrity.

d) Supports

Settlement under supports or anchor blocks due to soil consolidation, or erosion of foundation material, may lead to overstressing and failure. Progressive deterioration of older concrete may be due to AAR or freeze/thaw damage. Older concrete supports were not designed for high stress concentrations at the horns, and can result in deformation of the penstock, failure of the saddle or damage to the coating and lining system. When movement occurs at concrete saddles, the newly exposed contact area may be a source of corrosion. Wood cradles may show signs of crushing and decay. If buried plastic and fiberglass penstocks are not supported on a layer of sand, high stress points may occur above, or below, the shell surface due to objects such as stones or roots.

e) Joints

Under most circumstances, a joint is not functioning properly if water is seen leaking into or out of a penstock. Differential settlement can put stresses on a joint that it was not designed to handle, thus causing leakage. Cracked or flawed welds, and missing or badly corroded rivets or bolts, can be sources of problems on both the inside and outside of steel penstocks. Welding of riveted joints to stop leaks often results in the leak progressing along the joint as the weld advances. Concrete penstock joints, whether construction, expansion, contraction, or bell and spigot, generally use some type of waterstop or gasket which may deteriorate over time. Joints in wood penstocks are not usually problematic because of the water absorption capability of wood and thus swelling. Wood joints constructed using tongue and groove, splines, or butt joints allow some movement. Joints used to transition from wood penstocks to steel often leak due to deterioration of the joint packing material.

f) Alignment

Regardless of what material is used for the penstock, misalignment could indicate slope movement or settlement resulting in leakage or failure of the penstock shell.

g) Loss of Roundness

Several reasons exist for loss of roundness. These include excessive deterioration, under designed wall thickness, over or under compaction around buried, or partially buried, penstocks, and under design for surcharge or external loads. Partially or completely buried steel penstocks, supported on saddles, often buckle at the spring line due to deterioration and overstress. Unless under hydrostatic pressure, wood, plastic, and fiberglass penstocks are often out-of-round due to sagging.

h) Dewatering/Watering

Prior to maintenance, a watering and dewatering rate protocol should be developed and used. Excessive hydrostatic pressure on the outside of a buried penstock that has been dewatered could collapse the shell. When draining a penstock, excessive negative internal pressures could develop due to clogged or frozen air vents or valves. Concrete penstocks have a tendency to crack due to differential expansion when partially buried and dewatered. Wood penstocks should be de-watered slowly. If de-watering is necessary, wood penstocks should never be left empty for extended periods of time without maintaining a water spray internally and externally on the wood staves. Each time the wood staves are allowed to dry, they will lose some of the ability to absorb water, swell, and reseal any leaks. In addition, a possible loss of structural integrity may result from shrinkage.

i) Geotechnical

Unstable foundation conditions may be indicated by changes in the ground surface near the penstock construction; eroded soil or displaced rock at the bottom of slopes; and deformation or leaning of structures. Exposed penstocks are also vulnerable to impact from rockfall.

j) Vandalism

Exposed penstocks often act as targets and attract gunshots.

5.4.3 Corrective Measures

For the problems identified in Section 5.4.2, the following are common corrective measures:

- Maintain high quality and properly installed coating and lining on steel penstocks; keep spalls and cracked concrete repaired and sealed; keep wood penstocks watered as much as possible; and bury plastic and fiberglass penstocks for ultraviolet deterioration.
- Retrofitting a cathodic protection system could be more cost effective than excavating the entire length of a buried penstock to re-apply a coating system.
- Regular inspections of penstocks. In situations where breakdown of the coating and/or lining protection system has occurred, non-destructive testing (ultrasonic) can be used to assess wall thickness reductions. This could be done in conjunction with interior inspections to visually document the amount and depth of pitting.
- Monitor the supports for settlement, AAR, erosion around the foundations, and moisture problems at the concrete and steel contacts. Monitor wood saddles for deterioration.
- Mortar lining of smaller diameter penstocks.
- Acoustic monitoring of prestressed concrete cylinder pipe to listen for reinforcement failures.
- Drainage to improve slope stability or dissipate high pore pressures.
- Tendons and anchors to improve slope stability.

If any of the problems listed above are encountered, preventive measures should be taken immediately to avoid failure.

5.4.4 Opportunities

Based on condition assessments and optimization evaluations, the following could be considered as opportunities for upgrading to reduce head loss and extend the service life:

- Maintain exterior coating per coating manufacturer recommended to extend life.
- Reline interior per coating manufacturer recommended.
- Lining the interior of a severely pitted penstock can reduce head loss.
- Consider replacement of an expansion joint or flange that is deteriorated or leaking. Every small leak has a potential to cause erosion or settlement at the supports and a replacement may prevent a catastrophic failure.
- Repair or replacement of damaged or deteriorated supports, anchor blocks and bolts, wooden cradles, and steel bands may not extend life but could prevent a catastrophic collapse and failure.
- Total replacement of a penstock that has reached the end of its service life and no longer can be economically repaired.

5.4.5 Case Histories

No. 1 Wood Stave Penstock Deterioration (Replacement)
Bigfork Project (Pacificorp, 2001)

PacifiCorp owns the Bigfork Hydroelectric Project, located in Bigfork, Montana. When originally constructed circa 1909, the project included a 9-foot nominal inside diameter wood stave flowline (penstock). The flowline was constructed of fir staves approximately 6 inches wide, 4 inches thick, and 20 feet long. Steel splines held the ends of the staves in alignment while 5/8-inch diameter external steel bands, spaced 12 inches on center, held the wood staves tightly together. The flowline was assembled in an earthen trench and buried after completion, with a minimum cover depth of 3 feet, to a maximum of 9 feet.

The flowline functioned as intended for well over 80 years. It was dewatered on an annual basis to facilitate inspection and to conduct any necessary maintenance. As the staves aged, they began to show signs of deterioration, first near the ends of the butt jointed staves and more generally in the vicinity of the springline. Years of water and sediment passing over the butt joints eroded them, exposing the splines, and allowing soil material to infiltrate from outside the pipe. Voids formed from piping and erosion behind butt joints were common.

Over time, a gradual loss of support along the sides of the pipe combined with the weight of the overburden forced the pipe out of round and into a semi-elliptical shape. This created additional stress on the staves located near the sides and top of the pipe, resulting in crushing of stave edges, accelerated deterioration, and leakage in these areas. As the deterioration of the staves progressed, soil material began to infiltrate from behind the staves and into the flowline. This created voids and occasional sinkholes outside the flowline.

Increased concerns regarding noticeable changes to the shape of the flowline prompted efforts to begin documentation of the flowline's interior dimensions during annual inspections in the late 1980's. The flowline had station number tags lag-bolted to the wood. Width and height measurements, compass azimuths, and conditions in the flowline were recorded for year-to-year comparison and a guide for any future repairs.

The replacement of wood staves is a common practice for PacifiCorp at other facilities. However with the Bigfork flowline buried, the removal and replacement of staves was not considered feasible. Until 1991, maintenance typically included the installation of rectangular aluminum or steel plates backed with rubber gasket material. These plates were attached to the inside of the flowline using closely spaced lag bolts to cover areas where the wood staves had deteriorated and showed evidence of leakage.

The first 60 to 100 feet of the flowline experienced an accelerated rate of deterioration. This section had become significantly out of round, with inside cross section dimensions being reduced from a nominal 108 inches to as little as 94 inches. The poor condition of this portion of the flowline was believed to be due to the entrainment of air as the water entered.

Replacement of the entire flowline was considered in 1990, based on its deteriorating condition and the increasing number of liner plates required each year. However, economics did not support replacement of the entire flowline. As the first step of a life extension program, approximately 110 feet of the flowline was removed and replaced with a cast-in-place square box culvert section. This replacement alternative was selected based on an economic evaluation of various options. A flared section was constructed at the intake to make the upstream transition from a 9-foot diameter round intake to the 8-feet square box cross-section. An 8-foot long steel transition thimble was inserted into the downstream end of the flowline and cast into the concrete box section.

At the same time, efforts were made to repair other areas of localized deterioration within the flowline. Quick setting concrete cement was applied to relatively small areas of deteriorated staves on the inside of the flowline. The locations of suspected soil infiltration and sinkholes were established by driving a steel rod up from inside the flowline toward the ground surface. In some cases, removal of the rectangular plates revealed areas of completely deteriorated or missing wood staves. These areas were excavated from above, by hand, to expose the damaged staves. Repairs were made in these areas by replacing the metal liners, applying a concrete and bentonite slurry mixture to the outside of the flowline, and backfilling with pea gravel. Where needed, additional metal sheets were installed.

The life extension repairs described above served to extend the life of the wood stave flowline another 10 years. In 2000, the flowline experienced a major wall failure during the annual inspection. Considering the overall condition of the flowline and the magnitude of the failure, a decision was made to replace the flowline altogether. An evaluation of more than a dozen alternative materials resulted in the identification of a 10-foot diameter HDPE pipe product as the most economic material for the replacement.

The original wood stave flowline was excavated and removed. The new 10-foot diameter, 42-foot long sections of HDPE pipe were delivered to site from the manufacturer. The new pipe was laid in the excavated trench, and the joints were extrusion-welded together on site to form a continuous 1,689-foot section of flowline. Concrete transitions were constructed at each end, the trench was back filled, and the flowline ready for service in only eight weeks.

The application of this size of HDPE pipe for pressurized service is the first of its kind for the manufacturer. Due to the very low-pressure rating required for the Bigfork application, the HDPE flowline is expected to provide many years of satisfactory service.

No. 2 Wood Stave Penstock Deterioration (Replacement)
Victoria Hydro Project (MWH, 2001)

The Victoria Hydroelectric Project, owned by Upper Peninsula Power Company, is a 12-MW facility, 215-foot head, located on the Ontonagon River near Rockland in Michigan's Upper Peninsula. The existing 6,050-foot long, 10-foot-diameter wood stave penstock was originally constructed in 1927 and was rebuilt in 1959. The wood stave penstock begins at the steel thimble at the downstream end of the intake structure on the left abutment of a concrete dam, and roughly follows the left bank of the river channel, terminating at the steel thimble on the upstream side of the concrete thrust block at the surge tank. The wood stave penstock is above ground, supported on concrete saddles spaced approximately 10 feet on center. Wooden piles were driven to support the concrete saddles in fill areas. The horizontal and vertical alignment generally follows the topography of the riverbank, with numerous horizontal and vertical bends. There are excavation cuts into the hillside on the uphill side of the penstock alignment and some fill areas on the downhill side. An access road for penstock inspection and maintenance is located on the downhill side of the penstock between the penstock and the river.

The wood stave penstock consists of 3-1/2 to 3-5/8 inch thick Douglas Fir staves, pressure creosote treated, with cast iron Kelsey Butt joints. The steel bands, dating from the original 1927 construction, are 2-piece, ¾-inch diameter bands approximately 3 to 12 inches on center. Six vacuum breakers at various crown locations, three drain valves at various invert locations, several manholes, and numerous fire valve taps are installed along penstock. In addition to the steel thimbles at the dam intake structure and surge tank, five other steel thimbles existed along the penstock. The surge tank connection is located just upstream of the termination of the wood stave section.

An inspection of the exterior of the penstock found the wood to be generally in poor condition due to decay, crushing, angle shear, and delamination. Core samples indicated the wood to be in poor condition. Numerous saddles were found to be deteriorating and in need of repair to restore support and structural integrity, and drainage along the penstock route required improvement.

In summary, the existing penstock suffered from advanced wood stave deterioration, ovaling, settlement, and considerable leakage throughout its length. The condition of the penstock was not unexpected after more than 40 years of service. Continual maintenance and repair would be required to control leakage and keep the penstock functional throughout its remaining estimated two years of useful life. Because of the advanced state of deterioration, as a result primarily of its age, life extension options to enhance safety and improve performance were no longer considered economically viable. An investigation was deemed necessary to compare the feasibility and cost of different alternatives for a complete upgrade.

A number of alternatives were studied to determine the types of installations, alignments, and shell materials with which to upgrade the penstock. A new installation would either be above ground, buried, or partially buried. Regardless of whether the existing or new alignment was determined to be best, the new penstock would have to connect to the existing steel thimbles at the intake and concrete anchor block at the surge tank. The different material alternatives considered were another wood stave penstock, steel, pre-cast, or cast-in-place concrete, fiberglass, or high-density polyethylene.

As-built topographical surveying of the penstock alignment was performed to obtain relevant information on the topographical conditions affecting the replacement. Geologic mapping was also performed along the penstock alignment to identify geologic features and geotechnical conditions that would affect the evaluation of replacement options, including the potential for instability in the existing slopes and drainage considerations. A drilling and testing program was conducted at selected locations to identify foundation material characteristics relevant to the evaluation of alternatives and designs. Laboratory testing was also undertaken for classification, strength, and consolidation testing of the samples retrieved during the subsurface exploration program.

Based on evaluation of alternatives for the penstock replacement, a 9.5-foot inside diameter steel pipe, installed above ground, was determined to be the most feasible and economically competitive upgrade alternative. Because of the civil works required to offset a new penstock parallel to the existing penstock, the existing alignment was chosen for the new penstock, although the hydroelectric plant would be off line for an extended amount of time.

As an alternative to the frequent spacing of concrete saddles, a continuous free-draining ballast support for the pipe was developed. The continuous free-draining ballast support would eliminate the need for concrete saddles, avoid the need for pile supports, and distribute the load over a greater area, minimizing foundation problems.

No. 3 Steel Penstock Deterioration and Failure Schaghticoke Project (Brookfield Power New York, 2000)

The 14-MW Schaghticoke Hydroelectric Project, owned by Brookfield Power New York, was constructed in 1908 by the former New York Power & Light Corporation and consisted of a four-unit powerhouse located downstream approximately a half-mile from the reservoir. The water conveyance system consisted of a 1700-foot long canal connected to an 850-foot long, 12.5-foot diameter steel pipe ending at a 40-foot diameter surge tank. Originating inside the surge tank, four 280-foot long, 6-foot diameter penstocks are each connected to a generating unit.

During a normal shutdown of Unit 3 in 1998, the penstock for Unit 3 experienced a complete failure 30 feet above the powerhouse, as well as the scroll case manhole cover inside the powerhouse. This resulted in a stream of water under 150 feet head blasting the side of the powerhouse and eroding the surrounding embankments. The failed manhole cover resulted in as much as 4 feet of water flooding the interior, tripping the other operating units, and severely damaging almost all electrical and mechanical equipment.

It was two weeks before the eroded slopes around the powerhouse could be stabilized sufficiently to permit access and the search for cause of the penstock and manhole failures. Speculation arose as to whether geotechnical problems contributed to the failure, or whether it was the penstock material alone. Tensile and hardness testing and electron microscope examination of the penstock samples were conducted. A testing program of ultrasonic thickness measurements was established to examine the remaining three penstocks and main pipeline to the surge tank.

Figure 5.4-1 Schaghticoke Project
Uncontrolled discharge due to the failure of one of the 6 foot diameter penstocks upstream of the powerhouse
(courtesy of Brookfield Power New York)

Figure 5.4-2 Schaghticoke Project
Failed Section of Penstock
(courtesy of Brookfield Power New York)

Results showed that the failed Unit 3 penstock was severely weakened in two areas by localized corrosion. The routine shutdown of the unit caused a series of pressure pulses that ruptured the penstock at the weakened area. Flow through the rupture caused turbulence and resonance that resulted in cracking from fatigue, until complete separation occurred. The other three units were in good enough condition to return to service, but tests showed the 12.5-foot diameter pipeline needed significant repairs.

Because the possibility of another failure existed, complete replacement of the four penstocks and main pipeline, rehabilitation of the existing surge tank, and stabilization of the slopes underneath the penstocks were recommended. Alternatives considered for the new pipelines and penstocks included replacement with wood staves, fiberglass, steel, and concrete. Replacement with steel was chosen due to its overall cost being lower than the other alternatives.

Replacement of the pipeline, penstocks, surge tank, and related work resulted in an upgrade, thereby extending the service life of the water conveyances. The benefits included the ability to continue generating energy which typically averages 54,000 megawatt hours per year.

No. 4 Foundation Movement
Camino Project (MWH, 2000)

The Camino Penstock, located in El Dorado County, California, is 1,500 feet long, 10.5 to 12 feet in diameter, and connects to a powerhouse with two 75 MW turbines. The facility is owned by the Sacramento Municipal Utility District (SMUD).

The Camino Penstock has experienced downslope movements including both toppling and sliding of the folidated metamorphic bedrock since construction was completed in 1963. Over the years, Sacramento Municipal Utility District has contracted several engineering studies and made continuous adjustments to the penstock. Sections of the penstock have moved as much as seven inches along the 35 to 55 degree slope.

The stabilization system consisted of mass grouting the rock slope to create a "gravity structure" approximately 40 feet wide by 500 feet long. Using forty-four, 24-strand tendons, ninety feet long, the penstock route was stabilized against downslope and toppling movement by anchoring the "gravity structure" to more stable upslope bedrock. Additionally, the ring-girder foundations received anchor block support using tendons to resist their downslope sliding movements.

The complex subsurface conditions, steep slopes, and difficult access presented both engineering and construction challenges. However, the four-year stabilization project resulted in extending the service life of the water conveyance and continued power generation.

No. 5 Deterioration and Increased Headloss
Kingston Mills Project (EOP, 1995)

The project is located in Ontario, Canada and owned by Eastern Ontario Power. Project features include a 1.8 MW powerhouse and two 220-foot long penstocks (circa 1914, 1926, and 1975), of 6-foot diameter. The maximum static head is 40 feet.

An early 1990s inspection and condition assessment revealed wall thickness significantly diminished, and zebra mussel infestation was a concern. With the desire to keep the generating station serviceable, the owner faced a project feature at the end of its service life and with safety issues. One alternative was to remove and replace the 70-year old penstocks. However, this alternative could not be economically justified with only 1.8 MW of generation. New materials that increased the wall thickness were available and provided a possible solution.

The scope of the 1994 project included the relining of one of the 6-foot diameter penstocks. A second 8-foot diameter penstock was relined in 1995. The project specification outlined the use of a latex-based primer and topcoat (Wearlon, by Decora) to a coating thickness of 8-10 mils. The second relining project utilized an epoxy primer (Devoe 236) with added solids to fill any small irregularities caused by years of corrosion.

High-pressure water spray at 10,000 psi did not completely clean the surface of the first penstock, which included areas of 3/8-inch thick rust. A second high pressure soda blasting was used, which did clean the steel to an appropriate level. This work took approximately 3 weeks. To execute the subsequent (1995) project, the water pressure was increased to 20,000 psi, which reduced the surface preparation time to 1 week. The primer and topcoat was applied using a high-pressure spray system. Temperature and humidity were monitored to ensure conditions remained above the dew point. A Dry Film Thickness (DFT) of between 8 and 10 mils was required for coating. The second penstock was relined during mid-summer, which reduced the curing time of the primer coat to half a day, thus permitting application of the topcoats sooner. While no particular details are available on costs, improvements in surface preparation and time of application reduced the schedule for the second reline project.

Following completion of the first relining, the unit was tested and the output increased from 600 kW to 670 kW, an increase of 11.6%. GLP attributed 8% of the increase to the new coating properties, and the balance to other unit modifications. Testing of the second penstock after relining revealed only 1-2% increases in output. The speculation is that the smaller increase in output is likely due to the fact that the second penstock was oversized when built in 1926. The 1994 – 1995 epoxy reline project is estimated to have extended the service life of the penstocks by an additional 15 – 20 years, improved project safety, and reduced the zebra mussel infestation. Eastern Ontario Power had the added benefit of being able to improve plant performance.

5.4.6 Collective Knowledge

1. American Society of Civil Engineers (ASCE). (1998). *Guidelines for Inspection and Monitoring of In-Service Penstocks.* ASCE, New York, NY.

2. American Society of Civil Engineers (ASCE). (1995). *Guidelines for Evaluating Aging Penstocks.* ASCE, New York, NY.

3. American Society of Civil Engineers (ASCE). (1992). *Steel Penstocks.* [ASCE Manual and Reports on Engineering Practice : No. 79]. ASCE, New York, NY.

4. Electric Power Research Institute (EPRI).(2000). *Hydro Tech Roundup Report: Steel Penstock Coating and Lining Rehabilitation: Volume 3.* [TR-113584]. Electric Power Research Institute, Palo Alto, CA

5. United States Army Corps of Engineers (USACE). (1995). *Engineering and Design - Planning and Design of Hydroelectric Power Plant Structures.* [EM-1110-2-3001]. United States Army Corp of Engineers, Hyattsville, MD. http://www.usace.army.mil/publications/eng-manuals/em1110-2-3001/toc.htm

5.4.7 Technical References

Brookfield Power New York. (2000). Project files for Brookfield Power New York, Liverpool, NY.

UPPCO. (2001). Project files for Upper Peninsula Power Company, Houghton, MI.

5.5 Tailraces

5.5.1 Function

The tailrace essentially begins where the water exits from the turbine draft tubes, which, in many cases, coincides with the downstream face of the powerhouse, and continues to where the tailrace converges with the stream or river. In some cases when the powerhouse is located a distance from the river, the tailrace can be a manmade canal that connects the powerhouse discharge area to the river. Where the powerhouse is built immediately adjacent to, or in the river, the powerhouse can discharge directly into the river. The physical features associated with the tailrace can include natural embankments, man-made berms, concrete diversion walls, and excavations through overburden or rock.

Tailraces usually have an upwards slope from the invert of the draft tubes to the bottom of the river. Tailraces are typically designed for a velocity of four to five feet per second, receiving discharge from the turbine/draft tube which, for reaction turbines, can vary from eight to twenty feet per second or for impulse turbines can vary from 40-100 feet per second at the point or discharge, depending on the turbine size. The geometry of the tailrace as it merges with the river is sized to nearly match the river velocity when operating under normal conditions.

Some tailraces are tunnels serving underground powerhouses. A tail tunnel may have an enlargement just downstream from the draft tubes to accommodate surge associated with load acceptance. Tail tunnels experience the same problems as power tunnels, in general, i.e. instability, leakage, and differing site conditions. Rockfalls in tail tunnels have been known to block turbine discharge, leading to powerhouse flooding. Tunnel problems and their solutions are discussed in detail in Section 5.3.

Tail tunnels may experience the additional problem of depositioning a problem similar to tailraces. Sediment deposition during extreme events may cover the tail tunnel outlet, rendering it incapable of carrying turbine discharge.

The area immediately at the base of a dam's spillway is often referred to as a tailrace. This is a misnomer as the area at the base of a spillway is intended to dissipate energy rather than conserve, or recover energy, as is the function of the powerhouse tailrace.

5.5.2 Problems

The tailrace is a hydroelectric civil work feature that can be often neglected in the original design, especially when considering long-term maintenance and operation of the generating facility. Poorly designed tailraces can result in tailwater elevations that are significantly different from what was assumed in the original design. Tailwater elevations higher than expected will cause a reduction in the gross head resulting in reduced output and poor performance. On high head facilities this may not be a significant percentage of the gross head. However, on low head facilities, a small reduction in the gross head caused by higher than expected tailwater can have a dramatic negative impact on the plant economics.

For impulse turbines, the runner is set above the maximum tailwater elevation. If higher than design tailwater condition occurs, the tailwater can inundate the runner, requiring the unit to shut down. This can then lead to sedimentation within the tailrace accompanied by an extended outage to remove the accumulated sediment and other debris that could collect within the tailrace while the unit was shut down.

Excessively low tailwater elevations, even to the point where the draft tubes are exposed, and the draft tube vacuum is broken, can result in poor performance due to turbine instability, reduced output, and cavitation damage to turbine components.

After hydroelectric facilities are constructed, changes to tailwater elevations can occur over the life of the project resulting in reduced performance and output. These changes are due primarily to erosion and deposition within the tailrace channel or river downstream of the powerhouse.

a) Erosion

Erosion (or degradation) results when material is removed, or relocated, within the tailrace. This is typically caused by channel velocities that exceed the maximum permissible velocities for the material composition of the channel. For example, a tailrace channel consisting of an excavation through clay and silt material, without sufficient channel protection such as riprap, will result in the transport and erosion of the fines from the channel when velocities are high. Other facilities can be affected by erosion when downstream transport of fines continues after construction but the source of replacing this material is cut off by the construction of the dam. As the erosion or transport continues, the tailrace cross-section increases and water elevations reduce (see Figure 5.5-1).

If the tailrace channel is constructed from soils (excavated or diked), then care needs to be given to addressing the size of the rip-rap and stone armor. Size of the armor is dependent on the expected water velocities within the tailrace. Material that is sized too small for the expected velocities will be displaced and/or will result in significant erosion of the channel walls. This can lead to shoreline slope failures, relocation of materials, and an increase to the actual tailrace water levels.

Figure 5.5-1 Shoreline Erosion That Resulted From High River Flows (courtesy of Brookfield Power New York)

b) Deposition

Deposition results when material is deposited within the tailrace. This can be prevalent when the tailrace is immediately adjacent to the main river course. High flows have been known to deposit sediments and debris in the vicinity of the confluence of the tailrace with the river, causing an increase in the tailwater elevation and a reduction in unit performance and output. Depositions can also occur in rivers with high sediment loads. If the velocity in the tailrace channel is low enough, the sediments discharged through the turbines may settle out, forming sandbars and raising the water surface levels in the tailrace.

c) Deterioration

Some facilities have relatively short concrete walls (less than a few hundred feet long), known as training walls, separating the river from the tailrace channel (see Figure 5.5-2). Their original design function was to direct river flow away from the tailrace while taking advantage of the natural slope of the river to provide a lower tailwater elevation (and increased gross head) at the powerhouse and at the same time preventing debris from being deposited in the tailrace from the river. These walls are often neglected in the planning of maintenance programs, and when they deteriorate, water and debris can enter the tailrace, adversely impacting the tailwater elevation (and gross head) at the powerhouse.

Figure 5.5-2 Tailrace Training Wall Separating the River from the Tailrace (courtesy of Brookfield Power New York)

5.5.3 Corrective Measures

a) Degradation of Tailrace

A degraded tailrace can be corrected by placing non-erodable material within the tailrace to create a backwater effect at the powerhouse, and maintain unit performance and draft tube vacuum. Material can include medium to large riprap or concrete blocks. Survey data on the configuration of the tailrace and hydraulic modeling should be obtained as a means of determining the proper amount of material to be used, and the appropriate placement location within the tailrace channel. This will also frequently necessitate the addition of channel protection along the tailrace to avoid the erosion in the future.

b) Deposition within the Tailrace

Deposition can be corrected by removing the material that was deposited within the channel. This will usually require dredging the channel to the original design configuration. Since dredging can be costly, this option should be weighed against the value of the reduced performance and output. For minor deposition problems, it may be more economical to defer the dredging until such time as the cost will be justified by the benefits of the increased generation performance.

c) High Tailwater Elevations (impulse turbines)

For impulse units where high tailwater elevations cause operational problems or unit shut downs, a tailwater depression system can be installed. This system depresses the tailwater elevation below the runner using compressed air so as to avoid the operational impacts from high tailwater. See case history below for further discussion.

d) Other Causes

Repairing the concrete dividing walls that separate the tailrace and river should be carefully considered as these structures are generally difficult to access and repairs can be costly. Most construction work in the tailrace will usually require the use of divers, barges, or cofferdams, and result in lost generation due to the need for a plant outage. Because of the expensive nature of this construction, the benefits should be thoroughly evaluated to determine whether there is economic justification for the work.

5.5.4 <u>Opportunities</u>

The following are some opportunities that should be considered when planning tailrace modifications:

- When field modifications within the tailrace occur, such as adding stone armoring or excavation or dredging, a plant outage will likely be necessary to allow safe access and to avoid excessive turbidity. The owner should consider coordinating other outage related work such as inspections and repairs to minimize lost generation.
- In the event that a cofferdam is used and incremental costs are not excessive, consideration should be given to dewatering back to the powerhouse foundation. This will allow a thorough inspection of the powerhouse substructure and turbine draft tube areas that are not accessible under normal operating conditions.
- Care must be exercised when performing repairs to, or modifications of, a tailrace, so that changes implemented do not have an adverse impact on the velocity or water level in the tailrace.
- Tailwater depression systems increases plant availability and generation during high tailwater.

5.5.5 Case Histories

No. 1 Low Season Tailwater Level
NY Barge Canal Lower Mechanicville Hydroelectric Facility (NYSDEC, 1996)

The New York State Barge Canal is operated generally from May through November each year. During the winter months when the canal is closed to traffic, the water level is lowered in portions of the canal. At the Lower Mechanicville Hydroelectric facility the lowering of the canal water levels during the winter months reduced tailwater levels, causing turbine problems by the inability to maintain a vacuum in the draft tubes. This would ordinarily have resulted in shutting the entire facility during the winter months.

To compensate for lower tailwater levels during the winter months, precast concrete blocks (sometimes referred to as Jersey Barriers) were lowered into the tailrace just prior to the canal shutting down for the winter. The purpose of the blocks was to create a temporary tailwater pool, whereby the tailwater would be backed up to create a higher water surface elevation at the draft tube locations. This higher tailwater would be commensurate with canal elevations expected during the summer months. This relatively low cost solution allowed the hydroelectric facility to continue to operate throughout the winter months, increasing revenue and eliminating the need to provide powerhouse heat, and other expenses associated with winterizing the facility.

No. 2 Flood Erosion of Tailrace Embankment
Combie Project (de Rubertis, 1999)

The Combie Dam Project is located on the Bear River in the State of California. The project consists of a variable radius dam 762 feet long, with a maximum height of 100 feet and a powerhouse located on the left bank, just downstream from the left dam abutment. A 56 inch diameter pipeline provides water from the dam to the turbines. The powerhouse contains three vertical turbines each rated at 500 kw at 113 cfs. 48 inch diameter pipes from the penstock manifold supply each of the units. The powerhouse substructure is reinforced concrete, founded on rock, and the superstructure is concrete block. A short tailrace returns the discharge from the units to the river.

A flood occurred in 1997 that resulted in significant erosion along the south powerhouse tailrace embankment immediately downstream of the powerhouse. This section of shoreline also provided protection to the face of the powerhouse and the access road/parking area for the powerhouse. Several shoreline reinforcement alternatives were considered. However, placing grouted riprap was the obvious solution due to its lowest overall installed cost compared to more costly alternatives such as installing various forms of retaining walls. Grouted rip rap was installed along approximately 100 feet of shoreline. Construction was coordinated to coincide with low river flows in order to allow placement under relatively dry conditions.

Correcting this tailrace erosion problem allowed the continued operation of the Combie generating facility. Erosion that occurred as a result of the flood was addressed and the possibility of future erosion was mitigated by the additional grouting of the newly placed rip rap.

No. 3 Deteriorated Training Wall
Yaleville Project (Brookfield Power New York, 1982)

The Yaleville Hydroelectric Development is located on the Raquette River near Potsdam New York. The 700 kw facility consists of a dam 12 feet high 550 feet long, a powerhouse containing two vertical Francis turbines, and a concrete tailrace training wall measuring 9 feet high and 200 feet long.

The original purpose of the training wall was to provide for separation between the main river and the tailrace (see Figure 5.5-2). This wall kept debris out of the tailrace and also allowed for a lower than normal tailwater elevation at the powerhouse that produced a larger gross head for generation purposes. In recent years, the training wall was showing signs of significant concrete deterioration. There was of holes that penetrated through the entire thickness of the wall and concrete deterioration was evident throughout its length.

The owner considered two options for remediation. The first involved doing nothing and allowing the wall to continue to deteriorate. The second option required repair of the wall by overlaying the two faces and top with a 12 inch thick layer of reinforced concrete, followed by grouting the voids within the wall. The owner performed several hydraulic backwater analyses of the river adjacent to the powerhouse and tailrace to determine the effect of the tailrace wall on the tailwater elevations, and the resulting impact to annual generation. The analysis indicated that while the wall did help reduce tailwater elevations downstream of the powerhouse, the reduction in generation, assuming the wall continued to deteriorate, was not enough to justify the financial expenditure associated with repairing the wall. In addition, the owner did not see any negative impacts on the level of erosion or deposition that would warrant a significant expenditure to repair the wall.

Given the minimal operational consequences associated with the do nothing option, the owner decided to continue to defer any repair work on the wall until such time as operational problems could justify making these relatively expensive repairs. The owner determined that the operational benefits the tailrace wall was providing to the facility were insufficient to justify making repairs to the wall. The benefits of the analyses resulted in the owner's ability to re-allocate available funds to other higher priority needs for the facility.

No. 4 High Tailwater Level
New Colgate Powerhouse (Yuba County Water Agency, 2003)

The following case history is excerpted in its entirety from the Web Site of the Yuba County Water Agency:

> "The New Colgate Powerhouse, constructed between 1968 and 1970, is located on the Middle Yuba River at the upper end of Englebright Reservoir. The two generators are driven by vertical-shaft impulse (Pelton) turbines. The powerhouse is operated as a part of the PG&E power supply system.
>
> High tailrace water elevations, due to high flows in the river during flooding, reduce the space in the turbine runner pits. If the tailwater rises to the point where foam interferes with the rotation of the runner, a backsplash effect occurs in the buckets, resulting in irregular runner rotation, excessive turbine vibration, and instability of power output. To continue operation of the unit under rising tailwater level conditions, the water flow discharged through the unit must be reduced to reduce the amount of foam generation. If the tailwater level continues to rise, the units eventually have to be shut down, because they are not operable when submerged.
>
> The New Colgate Powerhouse Tailwater Depression element would avoid curtailed operations and enhance the ability to regulate flood inflow to NBB Reservoir. It would also increase the production of power and energy. During high flows a tailwater depression system would introduce compressed air into the turbine runner pit to depress the tailwater to a level that does not interfere with turbine operation, thereby allowing continued turbine operation and release of water from NBB Reservoir for flood space.
>
> During the original design of the New Colgate Powerhouse, installation of a tailwater depression system was contemplated. Even though the system was not installed, certain provisions were made for future installation of the system. These features included installation of a cantilevered deck for future installation of blowers, including blockouts for piping connections, a tailwater float well and gauge to measure and record tailwater levels, a curtain wall at the outlet of the tailrace conduit to improve air recovery, and miscellaneous embedded air and water piping. These features will be incorporated in the tailwater depression system design as appropriate.
>
> The tailwater depression system would include air compressors, air discharge piping with control valves, water-level sensors, power supply, and electrical controls. The air compressors would be of the high-volume,

low-pressure type, often referred to as "blowers." The blowers would be driven by electric motors.

Compressed air would flow from the compressors to a common pipe manifold, from where the compressed air would be distributed to two separate pipes, one to each runner pit. The existing ducts and openings for air lines would be used as much as possible. Control valves would be required in the air lines. Water-level-sensing instrumentation would automatically start the air compressors as the tailwater level rises above a pre-set maximum level, and automatically stop compressor operation when the level drops to the pre-set level.

Additional modifications would include a turbine shaft seal to prevent compressed air in the runner pit from traveling along the turbine shaft and blowing oil out of the turbine bearing, and extending the curtain wall at the tailrace outlet to minimize the escape of compressed air from the tailrace conduit. In addition, at the plant deck the hatch covers may have to be strengthened and sealed to be airtight.

Except during the period of high river stages, the powerhouse will be operated exactly in the same way as it is currently operated. The historical record suggests that, on the average, the system will be operated once every 2.2 years with durations ranging from three to twelve days and averaging six days.

The tailwater depression system will be designed to automatically turn on when the river flow is about 10,000 cfs. It would be designed to operate up to the peak stage on January 2, 1997, when the estimated flow in the Middle Yuba River was about 104,000 cfs. At higher river stages water levels in the turbine pit would rise as the air pressure from the compressor would no longer be able to fully offset the rising water and shut down if the stage is six feet higher than on January 2, 1997. A flood that would cause the river to rise that much would be a very infrequent event.

It is expected that the available space within the fenced plant area would be sufficient for laydown and staging of materials and equipment. All on-site work would be confined to the powerhouse, yard and existing excavated slop north of the powerhouse. No undisturbed areas are anticipated to be disturbed as a result of this project. Vegetation removal would be limited to that required to install four power poles on the slope north of and adjacent to the powerhouse.

Four items have been identified that would require a turbine shutdown:

- Installation of the turbine shaft seals.

- Completion of the installation of the air pipes connecting the existing open vents to the 30-inch-diameter air manifold.
- Installation of the pressure transducer on the blind-flanged 2-inch pipe outside each turbine pedestal.
- Installation of the 60 kV pole line where it crosses under the 230 kV lines (only the lower 230 kV line, which is powered by Unit No. 2, needs to be shut down).

The work on these items would be scheduled one unit at a time and at the time of a planned unit shutdown to avoid impacts on energy production. It is expected that the shutdown of each unit would have a duration of three weeks or less, with the controlling duration being the time required to install the turbine shaft seals. For each turbine unit, these work items would be performed simultaneously to avoid the need for multiple shutdowns. Advance planning, scheduling and coordination with the California ISO and PG&E would be required to avoid power losses.

The total estimated capital cost in Year 2002 dollars is $4,825,000. The average annual operation and maintenance cost is estimated to be about $27,200. This includes about $17,800 for energy consumption and lesser amounts for materials and supplies. It has been assumed that the operating and maintenance labor will be provided by the New Colgate Powerhouse operating personnel during their regular work hours. Therefore, no additional labor cost is anticipated."

5.5.6 Collective Knowledge

1. American Society of Civil Engineers (ASCE). (1992). *Guidelines for Rehabilitation of Civil Works of Hydroelectric Plants.* ASCE, New York, NY.
2. American Society of Civil Engineers (ASCE). (1989). *ASCE/EPRI Civil Engineering Guidelines for Planning and Designing Hydroelectric Developments, Volumes 1-4.* ASCE, New York, NY.
3. Creager, W.P. & Justin, J.P. (1950). 2nd Edition. *Hydro-Electric Handbook.* John Wiley and Sons, New York, NY.
4. Simon, Andrew L. (1981). 2nd Edition. *Practical Hydraulics.* John Wiley & Sons, New York, NY.
5. United States Department of Transportation. (1970). "Use of Riprap for Bank Protection" in *Hydraulic Engineering Circular No. 11.*

5.5.7 Technical References

Brookfield Power New York. (1982). Project files for Brookfield Power New York, Energy, Liverpool, NY.

de Rubertis, Kim. (1999). Project files, Cashmere, WA.

NYSDEC. (1996). Letter from the New York State Department of Environmental Conservation to Niagara Mohawk Power Corporation dated 1/18/96. FERC e-library web site. http://www.ferc.gov

Yuba County Water Agency. (YCWA). (2003). Project information, YCWA, Yuba County, CA. http://www.ycwa.com/mcolpp.htm

6.0 CHAPTER 6 – WATER CONTROL DEVICES

6.1 Introduction

Chapter 6 introduces the perspective of how real problems associated with water control devices are identified, defined and implemented to extend the life or upgrade a hydroelectric project or dam. Chapter 6 is organized by principal project feature, namely gates and gate hoists, valves and operators, and flashboards. Tables 6.2-1 through 6.4-1, located at the end of this subsection, summarizes by feature, the issues covered, opportunities for life extension and upgrade, and case histories for Water Control Devices.

The common approach to this guideline has been to document the numerous means and methods to upgrade, or extend the life of, civil works associated with a project, while acknowledging the selection of the most promising solution is often very project dependent. These guidelines are written as a reference tool to assist in this selection and not as a prescriptive design aid. Moreover, these guidelines are based on experience from existing works, and do not purport to cover all issues that could be associated with civil works.

The definition of life extension and upgrade as used in these guidelines is as follows:

Service Life Extension - Activity that extends the life of the civil feature beyond that which would be expected with normal maintenance (make it last longer).

Upgrade - Activity that improves performance of the civil feature beyond the current performance (make it work better). Exchange of a system or component with a similar or different system may not necessarily be considered an upgrade.

As background, Chapters 1 and 2 describe the processes to extend the life and upgrade hydroelectric civil works, and outline the steps to better understand the issues, identify opportunities, and recognize limitations. These chapters also provide insight on understanding the existing conditions and evaluating proposed changes that include identification and selection of the preferred alternative.

Chapter 3 is a review of innovative technologies that cover civil aspects of hydroelectric projects. These technologies have been developed mainly to reduce costs and improve profitability, reliability and environmental performance. Chapter 3 has been written around seven activities associated with civil works, whereas Chapters 4, 5 and 6 are focused specifically on the civil structures, water conveyances and water control devices, respectively.

This chapter describes examples and solutions as a way to illustrate techniques for improving the performance, or extending the service life of, the civil features of water control devices. Also provided in this chapter is general broad-based information, or "rules of thumb", to allow the reader to assess if, or when, a structure has reached the end of its service life, or if the structure warrants, or is capable of, improvements to its performance.

The common format used in Chapters 4, 5 and 6 starts with descriptions of the function of specific types of civil features, their problems and limitations, and possible corrective measures and alternative solutions for life extension or upgrade. Depending on the feature being described, not all aspects will contain the full range of information. Information on each feature is provided by subsection as follows:

a) Function.
b) Problems.
c) Corrective Measures.
d) Opportunities.
e) Case Histories.
f) References.

a) Function

A brief description of the function or purpose of the feature is provided. Some civil features may have multiple uses as part of the hydroelectric project. Depending on the actual use and functions of the structure, its expected service life and opportunities for improvement can be determined.

b) Problems

Typical problems and limitations (multiple if applicable), and the (root) causes are identified. If the civil feature has an operating system, a description of the limitations of the system is provided. Some problems are presented as a simple list and others are described in detail. General, broad-based "rules of thumb" regarding the service life, design or operational performance of a civil feature are provided as applicable. These are intended to provide guidance as to whether a feature has reached the end of its life, or the feature and its function could be improved.

c) Corrective Measures

Options are identified that have been used (or considered for use) to rectify the problems identified. For solutions with multiple alternatives, the pros and cons of the alternatives are discussed. However, the solution identified may be site specific, and may not be applicable to all similar problems.

d) Opportunities

Possible opportunities (additional benefits) are identified for upgrading (improving) the civil feature beyond that associated with just addressing a problem or limitation of service life.

e) Case Histories

Detailed examples are provided to describe typical solutions that have been used to address "real life" problems associated with that type of feature. In some of the examples the solutions may have extended the service life of a civil feature, improved its performance, or resulted in both an extension of service life and improvement of performance.

Each case history is structured to provide the following information:

- Background to the project and the function of the civil feature (background).
- Problems and causes (problem).
- Corrective measures and selected alternative (solution).
- Opportunities and benefits (results) provided to the owner as a result of life extension or upgrade.

f) References

There are three methods used in each Chapter to support the body of knowledge presented in the guidelines.

- Collective Knowledge.
- Technical References.
- General Resources.

Collective Knowledge is a collection of references considered by the guideline authors to be primary references that describe the function, operation, or design of each civil feature. Some of the references also provide information pertaining to inspection and assessment of the feature, problems, causes and possible solutions. The references are located at the end of each section to which they are pertinent.

Technical References are references specifically identified in the text of these guides, designated by name and date (i.e. ASCE, 1995) with the full reference at the end of each section.

General Resources include any background resource that is deemed to have value for broader reference and additional reading. It includes learned societies, government agencies and other sources. The General Resources Library is found in Appendix A.

Table 6.2-1 Gates and Gate Hoists

Issues or Problems Covered*

- Functional obsolescence/non-performance
- Gate failure
- Excessive corrosion
- Inadequate maintenance: personnel safety, excessive cost
- Loss of power
- Hoist motor overload and level switch failure
- Gate vibration
- Improper lake level systems
- Restricted gate movement
- Alkali-aggregate reactivity damage
- Seal rolling
- Bearing failure, greased and self lubricating
- Debris and/or ice jamming
- Discharging debris and ice

Opportunities for Solutions, Life Extension and Upgrade

- Ice removal systems: heat trace, air bubblers
- Rubber dam replacing gates or flashboards
- Replace or upgrade gate or hoist
- Replace gate seals and sealing plates
- Maintenance bulkheads
- Corrosion resistant bearings
- Coatings to prevent corrosion

* Additional information on Gates and Gate Hoists can be found in Appendix C, Table C-1.

Table 6.3-1 Valves and Operators

Issues or Problems Covered*

- Infrequency of use and overuse
- Severity of operating environment
- Variations in operating range and application
- Maintenance difficulties
- Remote operation and control issues
- Catastrophic failure
- Valve leakage
- Seals and bearing problems
- Corrosion/erosion
- Self-closing tendencies
- Deterioration and end of service life

Opportunities for Solutions, Life Extension and Upgrade

- Improved control of movement and range
- Decreased maintenance
- Increased reliability
- Replacement with different type of valve
- Improvements in hydraulic performance
- Application of abrasion and corrosion resistant valves

CASE HISTORIES		PAGE NO.
No. 1	Safety & Operational Concerns	340

* Additional information on Valves and Operators can be found in Appendix C, Tables C-2 and C-3.

Table 6.4-1 Flashboards

Issues or Problems Covered*

- Inopportune removal or failure resulting in loss generation
- Failure and damage from debris or ice
- Inability and increased costs to replace failed flashboards
- Over-designed
- Inadequate recreation and navigation depths
- Increased leakage and spillage
- Personnel safety during maintenance/installation

Opportunities for Solutions, Life Extension and Upgrade

- Increase revenues by decreasing spillage and maintaining head
- Improve personnel safety and reduced maintenance
- Enhanced environmental and recreational and operating conditions

CASE HISTORIES		PAGE NO.
No. 1	Flashboard Operation Misunderstanding	353
No. 2	Flashboard Failure Due to Ice and High Flows	354
No. 3	Personnel Safety, Lost Generation and O&M Costs	356
No. 4	Insufficient Spillway Capacity and Loss of Flashboards	359
No. 5	Over-Design of Flashboards and Failure of Spillway Crest	362

* Additional information on Flashboards can be found in Appendix C, Table C-4.

6.2 Gates and Gate Hoists

6.2.1 Function

Gates are used to block, control or regulate flows over a spillway, through a dam, outlet structure, or powerhouse. In controlling the flow in a river, the gates regulate and control water levels upstream or downstream of the hydroelectric project. Gates may be located on a dam's spillway(s), or in outlet structures to bypass flows around a hydroelectric plant, or in intake structures that release flow to the powerhouse and hydroelectric turbines. In addition to releasing flow, gates are also designed for use in releasing water-borne solids such as suspended sediments, trash, debris, and ice. Control or regulation of flow requires the gate to function as a metering device.

Valves can be used to perform the same functions as a gate, that of blocking, controlling or regulating flows. The primary difference between a gate and a valve is their location. Valves are located in, or at the end of a pressurized conduit, whereas gates are located in a free standing structure. Other differences between a gate and a valve have to do with their construction and the means of seating and support. Valves are discussed in Section 6.3.

A list of the various types of gates found and used at hydroelectric projects is shown in Table C-1 in Appendix C. The list includes possible uses for the different types of gates, along with the advantages and disadvantages of their use. Some of the gates listed may be considered antiquated, in that while there may be many gates of a type that are still in use, the gates have not been designed or manufactured since the 1960s. Gates fit into two discharge classifications: overflow (gates where flow passes over the gate, such as flashboards, fuse gates, drum, hinged crest, rubber dam), and underflow (gates where flow passes beneath the gate, such as radial, rolling weir, vertical lift). For a given size, underflow gates provide a higher discharge capacity than overflow gates due to the depth of submergence to the center of the underflow gate opening. Overflow gates are more suitable than underflow gates for sluicing floating debris with the minimal discharge of water.

Flashboards and rubber dams are simple gates that are installed on the crests of dams or spillways to provide a hydroelectric project with both storage and discharge capacity. While the advantages and disadvantages to the flashboards and rubber dams are shown on Table C-1 for comparative purpose with other gates, discussions of their operation, problems, and solutions are contained in Section 6.4 – Flashboards.

Hybrid gates have been used at some projects to provide the capabilities of both overflow and underflow gates. For example, small hinged gates have been added to the top of radial gates to provide the radial gate with the ability to sluice flowing debris as an overflow gate. Similarly, vertical lift gates can be designed for use as overflow gates for purposes of sluicing floating surface debris, wherein the gate is lowered vertically to allow discharge over the top. Generally, the hybrid gates are used when it is desired to have the capability of both underflow and overflow gates, but the dam site may not be able to support the use of both types of gates due to site constraints such as available space, alignment with river flow, or height of dam.

Gates may be sectioned or un-sectioned. Sectioned gates consists of sub-assemblies that are erected piece-meal when needed with stoplogs as a prime example. Stoplogs may be used as a means to block a discharge opening, with the individual stoplogs removed one piece at a time when flow must be released. Bulkhead gates are not a true type of gate but rather a classifying use of gate (as are the gates classified for use as intake, draft tube, spillway). Bulkhead gates and stoplogs are normally lifted vertically installed under no flow conditions for maintenance or emergency use, They often spend most of their life cycle in storage rather than in service. The location and function of a gate determines its design, method of operation, and service life.

The powerhouse usually contains gates or valves to control flow to the turbine, from the intake, and may have headgates and draft tube gates for turbine dewatering and maintenance purposes. In addition, the hydroelectric turbine may also contain wicket gates, which are not "gates" in the sense of the types of gates used on a dam. Turbine wicket gates are actually more closely related to a valve than a gate, as they have seating and supports and are located in a pressurized conduit. The turbine wicket gates are similar to the wicket gates identified in Table C-1, although the wicket gates used in a dam are operated under low heads (less than 30 feet) whereas turbine wicket gates can operate under heads that may exceed 1,000 feet.

Some of the gates listed in Table C-1 may be used at navigation locks and dams, or on tidal developments that include hydroelectric generation. Lock and dams, and tidal projects also utilize gates, such as the Miter, Vertical Sector, and Buoyant Flap gates that are not commonly associated with, or found at hydroelectric projects. Gates not commonly used at hydroelectric projects have not been addressed specifically in these guidelines.

Gate hoists are the mechanical device used to move the gate to allow the discharge of flow. Gate hoists may be fixed or movable, manually operated or motorized, manually controlled or automated. The different types of hoists that are commonly used to operate the gates, and the advantages and disadvantages to each, are described in Section 6.2.2c – Problem, Gate Hoists.

6.2.2 Problems

This subsection of the guideline differs slightly in format from other similar sections in that discussions of problems, causes, and solutions are presented together.

Problems associated with a particular type of gate are a result of the gate's function and operation in a wet environment, resulting in the deterioration of the gate or the gate structure. A major cause of problems with a gate and hoist system are obsolescence of function, operation, or safety, all of which define the service life of the gate and hoist system. A gate and hoist system is considered to be functionally obsolete when deterioration is compromising the system, or when the system does not have a discharge capacity that meets current regulatory needs. A gate and hoist system may be considered operationally obsolete if the gate can only be operated manually, the system cannot be controlled or operated remotely, or the gate must be tended to ensure safe or complete operation. Gate and hoist systems can be considered obsolete for safety purposes if the system is not safe to operate due to its condition; if the system must be tended to ensure safe and complete operation; if there are no means for dewatering the gate for maintenance purposes; or if there is insufficient room for access by personnel to operate or maintain the gate and hoist system. Infrequent operation is also a contributing factor that leads to the degradation and deterioration of a gate and its operability.

Smaller gates of wood construction have an expected service life of 25 - 50 years, depending on how much the gate is exposed to air, the type of wood used to fabricate the gate, and the number of wet-dry cycles to which the gate is subjected. Gates of cast iron and cast steel have also been used on hydroelectric projects, and their service life may reach 100 years or more depending on water quality and use of the gate. Gates have also been fabricated of aluminum and stainless steel, although those types of gates tend to be smaller in size or for special use purposes, and their service life is often governed not by deterioration but rather by the extent of damage they sustain during operation.

Most large gates of any type are made of fabricated steel (mild or high strength) and, with routine inspection and maintenance, the gate as a whole should have a service life expectancy of 75 or more years before need for replacement. Often after 75 years of service, the part of the gate most in need of significant repair (or replacement) is around the perimeter of the gate adjacent to the gate seals. Here a wet environment caused by seal leakage can promote rusting, and overall gate and concrete deterioration. Because at times the gate seals are inaccessible, the area around the seals is often not subjected to routine inspection, preventative maintenance (painting) or repair.

Many vertical lift gates can be raised clear of the water, and are easily inspected and maintained. Such gates that can be lifted free of the gate slots are the most commonly replaced because work can be done easily and economically. Removal of large gates such as radial, flap and drum gates, can be a costly undertaking. Many of the large gates are never removed from their installed position, and because the ends of the gate that run against a pier are difficult to access for inspection and maintenance, it is their condition that often dictates service life.

For gates with bearings (e.g. radial, hinged crest (motorized), and those vertical lift gates that use wheels or rollers) the bearing life may be significantly shorter than that of the gate's structural elements. This is dependent on the type of bearing materials used, type of lubrication materials and practice used, and environmental contamination of the bearing (leakage, sediments, chemical attack). In addition, the functional operation of the bearing may determine the early demise of a gate if increased bearing loads are experienced. An example of a bearing failure resulting in a failure of a gate was the 1995 failure of the tainter gate at Folsom Dam.

Conditions and problems associated with a gate, its guides, rails and seal plates, can be assessed if the various components can be readily accessed. This requires provisions for the installation of a bulkhead or emergency closure gate to allow dewatering, and a traveling or fixed hoist, incorporated into the design of the gate structure, to allow removal of the gate. Some hydroelectric projects do not include provisions for the installation of bulkheads, stoplogs or other maintenance gates upstream of the flow regulating gate, thus requiring the lowering of the pond, or the use of floating bulkheads, to dewater the gate. Lowering the pond may not be permitted for economic, environmental, or recreational reasons, and floating bulkheads may be the only option available for dewatering a gate. A floating bulkhead is installed upstream of the flow regulating gate, attached to piers and positioned and erected by barge. Once erected, the bulkhead assembly does not move, and the term "floating" only refers to the means of positioning and erection.

Most gate failures, as identified by the USSD (USSD, 2002), are reported as having occurred due to one or more of the following causes:

- Excessive corrosion caused by inadequate maintenance painting, lack of cathodic protection, and improper material selection.
- Overall inadequate maintenance including seals, lubrication, lifting cables, gearing, hydraulic systems, and electrical components.
- Loss of primary power supply and lack or loss of back-up power supply.
- Overtopping of gates.
- Debris jamming gates.
- Gates discharging debris.
- Hoist motor overload caused by insufficient hoisting capacity.
- Gate vibration.
- Improper installation and or use of automatic lake level systems.
- Lack of periodic exercising of gates and gate operating equipment.

The problems, causes, and solutions presented in the guidelines are considered to be representative and common to most gates. The problems that occur with any particular gate, other than design related, are the result of gate function, installation and conditions to which the gate is subjected including operation and maintenance. The problems that are described in the sections to follow are not in any particular order of importance or frequency of occurrence. The solutions presented are intended to extend the service life of the gate and gate hoist system. The problems, causes, and solutions exclude further discussions of obsolescence of function, operation, or safety. The solution for the problem of obsolescence is upgrading the gate and gate hoist system to meet project requirements.

Common problems presented in the following section include:

a) Restricted Gate Movement.
b) Gate Seals.
c) Gate Hoists.
d) Gate Vibration.

a) Restricted Gate Movement

Gates should move smoothly, without hesitation or jerky motion. Gates that do not move smoothly could cause overloading and damage to the gate, the gate's connection to the lifting hoist, the hoist itself, or the anchorage and structural supports or support structure for the hoist. In addition, operating personnel do not have much confidence in a gate that does not operate smoothly, nor should such a gate be operated remotely without personnel to monitor its movement. Some of the causes and corrective solutions are presented for the following common problems:

i) Motor Overload.
ii) Expanding Concrete.
iii) Seal Rolling.
iv) Unequal Length of Chain or Cable.
v) Unequal Loading of Chain or Cable.
vi) Debris Wedging.
vii) Bent and Damaged Gate Members..
viii) Bearings.

i) Motor Overload

Motor is overloaded resulting in a non-uniform transfer of torque into the hoist's gearbox. Overload may possibly be due to: under sizing of the motor, additional frictional or resisting gate loads, problems in the hoist's gearbox, misalignment of the drive shaft(s) or reduction gears, seized bearings, loss of an electrical phase, old age and deterioration of the motor windings.

Solutions:

- First, determine if the problem is electrical as related to the motor. If the problem can be addressed by installing a new motor of equal size, unless the motor was determined to be undersized for current operating requirements, then that is the simplest and most economical solution.
- Motor overload may be a seasonal problem caused by the additional weight on the gate from the accumulation of sediment and debris or ice. Solutions to address the seasonal weight problem may include more frequent gate operation to flush sediment and debris, or replacement of leaking gate seals to reduce ice accumulation, or the installation of gate heaters to prevent the gate from being frozen in the gate slots or the seals freezing to the sealing plates.

- Second, assess the condition and workings of the hoist, reduction gears, drive shafts, bearings and the remainder of the drive train. Except for motor replacement, repairs to the hoist's drive train are often the next least costly solution if the repairs take care of the problem.
- If the problem cannot be shown to be related to the hoist, assess the gate to determine the cause of the additional loads that the hoist must lift.

ii) Expanding Concrete

Alkali silica reactivity and alkali aggregate reactivity are chemical reactions in the concrete that cause the concrete to expand, reducing the clear opening and resulting in binding of the gate in the gate slots or gate structure. Expansive concrete can cause problems with any type or size of gate, but is especially a problem for tall gates, or large gates such as radial, drum, and hinged crest gates that require close tolerances to the supporting structure for water tightness.

Solutions:

There are limited solutions to resolve problems with gate operation that are caused by expansive concrete. The solutions do not address or correct the problem of expansive concrete, rather they provide a way to restore the operation of the gate.

- Remove concrete from behind the gate guide to restore the clear opening.
- Cut slots in the concrete to relieve the binding pressure on the gate. Due to the continued expansive growth of the concrete, slots may have to be cut a number of times over the life of the structure.
- Trim the edges of the gate to restore proper clearance.
- Replace the gate, and incorporate into the replacement gate's design the means to allow further and future adjustments to the "width" of the gate.

iii) Seal Rolling

Seals can roll over (usually when lifting a gate) and wedge the gate between the sealing surfaces, thereby damaging the seal and increasing the lifting loads to be overcome by the hoist.

Solutions:

Replace gate seals and redesign the means used to attach the seal to the gate or use a different seal geometry. See Section 6.2.2.b – Problem, Gate Seals, for additional information.

iv) Unequal Length of Chain or Cable

For those gates operated by paired lifting chains or cables (also referred to as wire rope), unequal lengths of chain or cable will cause the gate to be lifted unequally, resulting in the twisting and binding of the gate in the gate slots or guides. Cables of unequal length could be caused by overload stretching or improper length when installed. For additional information on problems associated with gate hoists, see Section 6.2.2.c – Problem Gate Hoists.

Solutions:

Most gates have turnbuckles or other mechanical means to equalize the lengths of the lifting chain or cable. Adjust accordingly.

v) Unequal Loading of Chain or Cable

The lifting chain or cable could be loaded unequally, resulting in the twisting and binding of the gate when it is being operated.

Solutions:

- Check to determine if the hoisting equipment is lifting the cables or chains at equal rates. One of the hoist drums may lag the rotational timing of the other drum due to misalignment, problems with the gearboxes, or torsional twisting of the interconnecting drive shafts.
- Check for binding of the gate on one side, resulting in unequal loading on one of the hoist cables or chains.

vi) Debris Wedging

Debris is stuck between the gate and support piers or guides, causing binding of the gate.

Solutions:

- Modify the gate to prevent debris from becoming wedged between the gate and gate supports. Modifications could include extending plates from of the upstream side of the gate to reduce the width of the gap between the gate and the support piers or guides.
- Vertical-lift gates in low head outlets, where the top of the gate is close to the water surface when raised, tend to entrain debris down past the top seal due to the gap provided by the set back of the sealing face at the top of the discharge opening (Figure 6.2-1). Solutions to prevent entrainment include redesign of the gate seal or reducing the potential for debris entrainment are presented in Section 6.2.2.b - Problem Gate Seals.

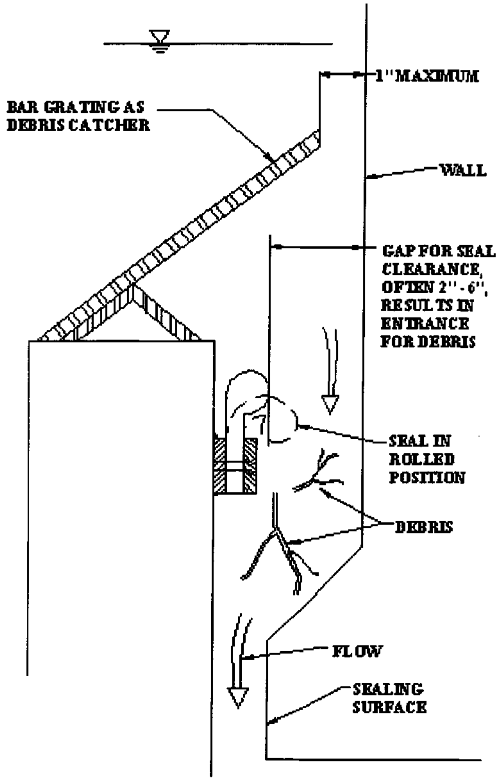


Figure 6.2-1 Debris Catcher

vii) Bent and Damaged Gate Members

Bent and damaged gate members could cause twisting of the gate, resulting in the gate not being lifted smoothly. Members could be damaged by deterioration, ice loads and accumulation, impact by debris and ice, either on the gate, or when discharged over the top of the gate, or structurally overstressed during operation.

Solutions:

- Inspect the gate for any damaged members and replace as necessary.
- Ice loads on a gate may be minimized using deicing methods described in Section 6.2.2.b.iii.

viii) Bearings

Increased friction in the bearings of a wheeled or roller gate, which are often submerged, or in the trunnion bearing of a radial gate, which is exposed to the elements, will increase the loads on the gate hoist. Properly greased bronze bushings have a friction factor of 0.15 if they are periodically exercised, but if the bearings are corroded, or were poorly lubricated, or an unsuitable grease was used, the friction factor could vary from 0.3 to 0.5. For assessing the design of a gate with lubricated bearings, the FERC and the USACE suggest using a friction factor of 0.3. Self-lubricated bronze bushings on stainless steel pins have a friction factor of 0.1, and lead-babbitt bearings on mild steel have a friction factor of 0.2 to 0.3.

Solutions for lubricated bearings:

- Review lubrication procedures, length of lubrication lines, frequency, and timing of when the lubrication is applied, to determine if the lubricant is reaching the bearing. This is especially true for bearings where the lubricant is pumped into the bearing and maintenance personnel cannot monitor where the lubricant is going. The 1995 failure of a tainter (radial) gate at the Folsom Dam was attributed to an increase in friction in a trunnion bearing, which resulted in the overloading and failure of a secondary bracing member in the gate arm. The failure led to the FERC's 1998 Tainter Gate Initiative, which required the inspection of all tainter gates, and analysis of bearing friction and stresses in the gate arms. As a result of the initiative, it was discovered that even the assumed maintenance-free, "self-lubricating" bearings required inspection and possible maintenance.
- Review the adequacy of the lubrication procedures and lubrication materials, and modify as appropriate. Determine if the lubricant is coating the bearing, especially for radial type gates where the trunnion bearing may rotate less than 90 degrees.
- Ascertain that maintenance personnel are not "cutting" the lubricant with a petroleum, or other liquid material, to improve the viscosity of the lubricant and its ability to deliver the lubricant to the intended location.
- Disassemble and investigate the condition of the bearings, repair or replace as necessary.

Solutions for non-lubricated bearings:

- Disassemble, if possible, and investigate the condition of the bearings, repair or replace as necessary. Problems have been recorded with the use of self-lubricated bronze bushings on radial gate trunnion bearings, where bird droppings have caused expansion of the graphite plugs, resulting in damage to the bushing and thrust washer. Other problems include water absorption of the plugs, resulting in seizure of the bearing. External indicators of such a problem may include cracking or damage to the trunnion thrust washer.

b) Gate Seals

The purpose of a gate seal is to close off the open gap between the edge of a movable gate and a fixed sealing surface. Today, for most types of gates at a hydroelectric project, seals are made of rubber. For smaller gates manufactured of cast iron or cast steel, the seals are often of dissimilar metals such as stainless steel and marine bronze. Gate seals have also (and sometimes still are) been made of wood or plastic, although these are usually limited to gates under low heads (less than 50 feet). Leather, rubber, wood, or plastic seals can be found on all types of radial, vertical lift, or maintenance bulkhead gates.

The primary problems that occur with gate seals are usually associated with excessive leakage, improper installation (inappropriate use of a particular seal geometry or means of attachment), or damaged or worn seals. Excessive leakage not only results in loss of water, it can lead to erosion of the surfaces on which the seals rub, or erosion and damage to concrete surfaces. In freezing climates leaking seals can result in the build up of ice on the downstream side of the gate, and could prevent a gate from opening due to the added weight of the ice on the gate, or the freezing of the gate to the adjacent side surfaces. Generally, the volume of leakage past rubber seals is in the order of 0.01 gallons per minute per foot of wetted perimeter (gpm-ft), and for metal on metal seals, the allowable rate of leakage is 0.1 gpm-ft (AWWA-C-50).

The causes of worn or damaged seals, with the focus on rubber seals, are discussed in the following categories:

i) Worn Seals.
ii) Damaged Seals – Debris and Flow.
iii) Damaged Seals – Ice.

i) Worn Seals

Rubber gate seals are usually made from a flat strip of rubber, or shaped by a molding or extrusion processes. There are many types of seal shapes that have been used on gates, with an effective and popular shape being the J-bulb or music note seal (Figure 6.2-2), in either the solid or hollow bulb form. Advances in seal design and manufacture since the 1950's have resulted in seal designs that overcame the limitations of J-bulb seals and flat rubber sealing strips as commonly used in pre-1950 gates. Current seals (bulb seals or otherwise) and their mounting attachments are adjustable, providing more flexibility (movement) in the seal, and are capable of resisting water pressures from either side. Bulb seals work best when allowed to deflect rather than compressing the bulb against the sealing surface.

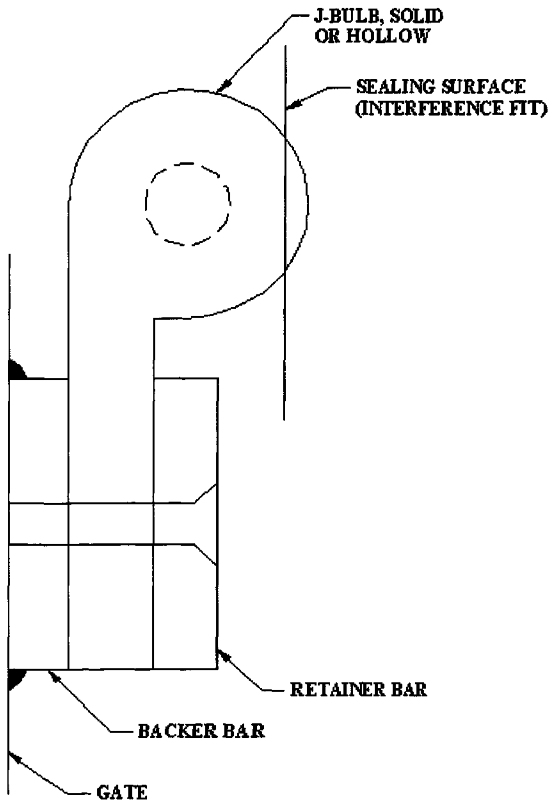


Figure 6.2-2 Gate Seal, J-Bulb

Rubber seals wear as a result of rubbing against an abrasive sealing surface, or by abrasive materials trapped between the seal and the rub surface. Gate seals may rub against sealing surfaces of concrete, wood, or steel. Concrete and steel surfaces must be smooth, burr-and rust-free to prevent wear and damage to the seals. For steel sealing surfaces, a stainless steel overlay or cladding is often used to provide the seal with rust-free sliding surface. Concrete sealing surfaces need to be ground to a smooth surface, because any leakage past the seal will eventually result in the erosion of the concrete surface. Worn seals and deteriorated sealing surfaces will result in increased loads that will have to be overcome by the gate operator.

Hard wood "seal plates" have been used with low head applications, and the tightness and direction of the wood grain is important to minimize erosion due to seal leakage. Wood seal plates that are continually exposed in a wet-dry area will experience accelerated decay, whereas wood seal plates that are submerged may have a useful service life that approaches one hundred years.

A seal made from a flat rubber strip is the simplest of all the different types of rubber seals, and is not sensitive to dimensional change except by abrasive wear. The rubber strip is simply bolted onto the gate and laid up against the sealing surface. The solid bulb seal is molded, and like the flat strip, the dimensional shape of the solid bulb seal is not sensitive to change except by abrasive wear. In contrast, the hollow bulb seals are more flexible and will deform, thereby increasing the seal's contact with the sealing area. The deformability of the hollow bulb seal is also a drawback because the seal tends to go out of round or to flatten when kept in the same stressed position for an extended period of time, as would occur when in contact with a fixed surface. Hollow seals are manufactured by an extrusion process and the seals are often received at a site already out of round due to the seal being coiled or rolled up for shipping. Hollow bulb seals along the vertical edge of radial gate have been observed to split through the bulb (starting at the top free end of the seal), due to tension created in the bulb from the friction of the seal sliding vertically along the sealing surface.

At the time of manufacture, the rub surface of a rubber seal can be clad with a fluorocarbon (usually green in color) to reduce the sliding friction, and extend the wear life of the seal. An unclad solid rubber bulb seal should last 5 – 20 years depending on the abrasiveness of the sealing surface on which the seal rubs, the frequency of gate operation, and exposure to ultraviolet light. Fluorocarbon (PTFE) clad seal should last at least 5 years longer, and the PTFE cladding can also reduce seal friction by 80 – 90%.

Wood and plastic gate seals are usually shaped as blocks that are bolted to the gate, and allowed to rub against a sealing surface which is often concrete. The service life of a wood seal is 5 – 20 years, depending on the roughness of the concrete, seal thickness and its exposure to wet-dry conditions, the hydrostatic pressure against the seal, and the frequency of gate operation. The service life of a plastic seal is 25 – 50 years and is totally dependent on the roughness of the concrete, the hydrostatic pressure against the seal, and the frequency of gate operation.

Metal seals of dissimilar non-corrosive materials should have a life expectancy of 75 or more years. The life of the metal seal will be dependent on the hydrostatic pressure against the gate, and the potential for sediments to become trapped between the seal surfaces.

ii) Damaged Seals – Debris and Flow

Seals can be damaged by debris, flow past the seal, or by the rolling of the seal, due to poor means of attachment or seal geometry. Debris can become wedged between the seal and the sealing surface. For the side and bottom seals of a gate, any debris that is located between the seal and the sealing surface will result in increased leakage, but if the seal is properly attached, the debris should not result in seal damage (see Section 6.2.6, references 5, 18, and 19 for typical attachment details). Any seal that rubs against a sealing surface has a tendency to roll due to debris or water flowing past the seal (see rolled seal on Figure 6.2-1) and will likely be damaged and may be torn from the gate.

The top seal of a submerged, low head gate is most susceptible to damage by debris, especially for those gates where the top of the gate in the raised position is within three feet of the level of the water surface. Due to the distance between the seal, the sealing surface, and the face of the gate, debris often wedges between the top seal and the rub surface due to the downward flow of water past the seal (Figure 6.2-1). The distance between the seal and the sealing surface can also allow the top seal to roll over, and possibly be torn from the gate, due to the velocity of the water past the seal. Figure 6.2-1 also shows a way to modify the top of a low head gate to prevent debris from being wedged on the downstream side of the gate between the seal and the sealing surface.

Seal geometry and means of attachment to the gate can be selected so that the seal is not susceptible to being rolled over due to the velocity of water past the seal, or due to the wedging of debris between the seal and the sealing surface. However, the seal could still be damaged by debris wedged between the seal and the sealing surface when the gate is operated (Figure 6.2 - 3). The double stem top seal (often referred to as an "inflatable seal" due to the means to equalize the pressure behind the seal) is highly recommended for most applications, whether it is located on the top or sides of the gate.

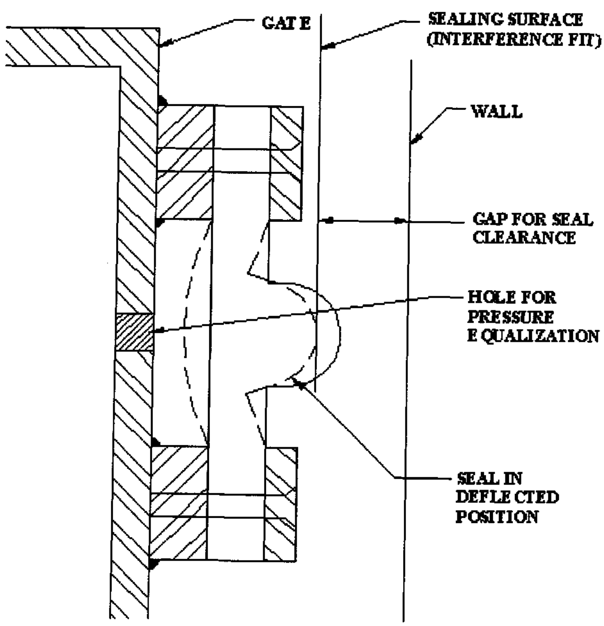


Figure 6.2-3 Gate Seal, Twin Stem

iii) Damaged Seals – Ice

Ice can damage gate seals by allowing the seal to be locked into the ice and frozen to the sealing surfaces, resulting in the seals being possibly torn off if the gate were operated. In addition to the damage to gate seals, vertical lift gates are susceptible to being frozen into the gate slots during sub-freezing conditions. This is due to the conductivity of steel and concrete which allows the stagnant water in the slot to freeze, locking the gate tightly in the slot. For new construction, heaters are often installed behind the sealing surface to prevent the seals from freezing.

Gate heaters were initially constructed using heated oil circulated through tubes embedded behind a steel sealing surface. Due to environmental concerns this method is not widely applied, or as a minimum, uses only a biodegradable oil. Electric resistance heaters have also been used, with the heating element being installed in air in a tube behind the steel sealing surfaces. Resistance heaters are costly to operate and maintain, as low level has power to be applied to the element, even during non-freezing months, to prevent condensation within the tube and corrosion of the element. Because the elements were installed in one piece, the resistance heaters are extremely difficult to maintain and are not conducive for use as long or curved elements.

Electric heaters of the bar or mineral-insulated (wire) type are the least expensive and easiest type of heater to construct, install and operate. The low wattage heaters are often immersed in an environmentally safe glycol, set in a tube behind the steel sealing surface. Due to its small size and flexibility, the mineral-insulated type can be installed in very long lengths and tight-bending radii.

There are numerous other types of exotic heaters that have been used such has freon, but while energy-and usage-efficient, they have not been proven effective or safe, due to environmental concerns or worker safety issues.

For existing gates without heaters, air bubblers, submersible water heaters or pumps can be used to circulate warmer water around the seals to prevent their freezing to the sealing surfaces, and to prevent a gate from being frozen into the gate slot. Steam lances can also be used to inject steam or hot water into the gate slots and onto the seals, to free them from ice prior to opening the gate. Installing shields to prevent wind from blowing sub-freezing air onto the downstream side of the gate seals will also help to reduce seal freezing. If it is desired to use electric heaters, then modifications must be made to the sealing surface (if there is sufficient room, and the heater tubes can be protected), or the sealing surface has to be replaced to accommodate the heaters. Minimizing gate leakage is the best solution to minimize gate freezing and damage to gate seals. Circulating water in the gate slot is the best solution to prevent an existing gate from being frozen in position.

For existing gates without heaters, frequent gate operation (jogging up and down in small strokes) will prevent a gate from being frozen into the sealing surfaces or gate slots. If seal leakage is excessive, a massive ice block may form on the downstream side of a gate, requiring that the gate be opened to loosen and flush the block away. If ice blocks are allowed to form on the downstream side of a gate, they will eventually prevent a gate from opening, especially hinged crest gates, or flashboards. If the gate discharges into a passageway, an ice block could lock and freeze the gate in the closed position.

c) Gate Hoists

The purpose of the gate hoist, or gate operator, is to raise and lower a gate under controlled conditions. When subjected to proper and routine maintenance, gate hoists, in theory, have an unlimited life. Unless a hoist has been operated beyond its rated capacity, an upgrade or replacement is often done as a convenience, particularly if it provides better operational control of the gate, or allows untended operation, or replacement is due to lack of replacement parts. As with gates, insufficient or lack of operation are contributing factors to the degradation of a hoist and its operability. As pieces of machinery, gate hoists do not operate at high speeds (movement of large spillway gates is commonly less than 12-inches per minute), nor are they subjected to many hours of operation. A number of the problems associated with gate hoists are due to their lack of use, or operation for a limited range of motion. Often the life of a gate hoist is determined by operational improprieties, such as operation of the hoist beyond its capacity, that either shorten its service life or result in a failure.

Gate hoists may be mechanical (electrically or manually driven), hydraulic (oil or water) or pneumatically operated. Buoyantly operated gates, such as self-operating hinged gates, and hydraulic systems using water as the operating fluid, are antiquated systems that are no longer commonly found. The guidelines do not address antiquated systems unless the system is still commonly in use at hydroelectric projects.

If there are significant problems with a hoist in its reliability or ability to raise or lower a gate and these cannot be corrected by repair or minor modifications, the best solution may be replacement of the hoist. In order to evaluate a life extension or upgrade of a gate hoist, the evaluator needs to understand the strengths and weakness of each of the different types of hoists to determine their applicability for the intended use.

Lifting a gate using two pick-points is common, especially when the ratio of gate width to gate height is 0.75 or greater. It is the rule rather than the exception, that the center of gravity of a gate is not equidistant from pick points, except on paper. This translates to unbalanced loads on paired rigging systems. Ballast can be added eccentrically to a gate to shift the location of the estimated centroid to counter such unbalanced loads.

Measurement of differential loads during lifting or seating of a gate of known weight and centroid position, can assist in diagnosing growing problems with sliding and rolling resistances that are not generally observable due to the gate's submergence.

Close inspection of a raised (open) underflow gate may indicate the gate is in good condition and its weight is balanced, thereby suggesting that the problem may be defects in the submerged guide, rail, seal, or seat plate systems. The visual observance of the routine operation of a gate, in combination with data obtained by manual or automated means, will provide insight as to the location of a problem in the operation of a gate hoist or the gate itself. Tracking dry and submerged weights of gates is not a common practice, but the potential exists, and such information may be helpful in monitoring the operation or condition of a gate. Currently, the most accurate method to track loss of mass of features that deteriorate imperceptibly is the loss of weight. Incorporating strain gages into rigging such as lifting cables, stems, gate arms, and latching beams, should not be difficult relative to the load distribution information obtained.

If the gate hoist is driven by electric motors, attention must be given to both the primary and backup power supply. If power (especially backup power) is not available to the motor driven hoist at all times, then there is no assurance that the gate can be operated to release and control flows at a time of need. While many gate hoists have the means to be operated using portable (low-voltage) power tools or energy packs, or by manual cranking, their operation may be slower, time consuming and often insufficient for the needs of the timely operation of the gate(s).

This section of the guideline discusses common types of gate operators, their limitations and problems. By understanding the limitations of the operating equipment, a possible cause of the problem can be identified and a solution can then be designed. Gate operators typically used at hydroelectric projects, and their common problems, causes, and solutions, include:

- Rack and pinion operator.
- Screw stem operator.
- Hydraulic actuators.
- Hoists or winches, connected to the gate by:
  - Cable (also referred to as wire rope).
  - Chain.
  - Roller chain.

A common problem in the sizing of any type of gate hoist is failure to address the hydraulic downpull that can occur on a gate: a force that results in a significant increase in operating loads on the hoist. The downpull force generally occurs due to the reduction in pressure (suction) caused by the velocity of the discharge under a gate, with downpull forces reducing substantially after the gate has been lifted a height that is 3 to 5 times the thickness of the gate. Downpull also occurs as a result of forces developed by flow discharging over the seals at the top of a gate, as typically occurs with submerged gates. The magnitude of the downpull force on the bottom of a gate is a function of the shape of the bottom of the gate, whereas those on the top of the gate are a function of seal projection and clearance around the top seal. In applications where a gate is located mid-tunnel, as the gate lip approaches the top of the tunnel, the hydraulic downpull force on a gate may reverse and occur as an upthrust on the gate. Normally downpull and upthrust forces are addressed and accounted for in the original design of the gate and gate hoist, but if either the gate or the hoist is to be, or has been, replaced, then the magnitude of the downpull and upthrust forces may have to be readdressed. For information regarding the estimation of downpull forces, and how the forces are affected by the shape of the bottom of the gate or the projection of top seals, see the technical reference (Lewin, 2001), listed at the end of this section.

Some gates can be designed as 'fail safe', with 'fail safe' being defined as that location to which a gate will position itself, either automatically or manually, to control the release of flow in a manner that ensures the safe operation of the hydroelectric project. The fail safe position varies with gate function, as follows:

- Spillway gate (discharge or control): Open, to maintain flows and prevent increase in headpond level.
- Intake gate: Closed, to prevent uncontrolled release of flow.
- Emergency closure: Closed, to prevent uncontrolled release of flow and loss of headpond.
- Sluice: Closed, to prevent uncontrolled release of flow and loss of headpond.

Gate hoists are key to determining the fail safe position of a gate. Certain types of hoists are not suitable for allowing a gate to move into a fail safe position, primarily because the hoist requires an external source of power to operate. Generally, hoists such as rack and pinion, screw stem, roller chain, or very large cable hoists all need power to operate. Hydraulic hoists, and smaller cable and chain hoists, can be designed to allow the gate to move by gravity without the need for an external power source. Often gates are allowed to move to the fail safe position upon loss of power, either automatically or by manual activation.

i) Rack and Pinion Operator

Rack and pinion gate operators are one of the earliest (pre-1900) types capable of providing a force for raising or lowering a gate. The operator consists of a geared rack attached to the lifting stems of a gate, with a pinion on a fixed shaft which, when rotated, causes the rack and gate to move. Rack and pinion operators were commonly found on vertical lift gates, although they have been used on small radial gates, and the rack and pinion operation is the principle for the operation of the rolling weir gate. A rack and pinion operator can be used on wide gates by using multiple stems to provide the forces needed to raise and lower the gate.

Each gate has its own rack and pinion operator, including any mechanical gears needed to develop the mechanical advantage and required operating forces, but each gate may not necessarily have a dedicated source of power. Often power was provided by "manpower", or the motorized power source was portable, or moved from operator to operator. Many sites used a single, motor driven line shaft to operate multiple gates, one at a time, by the use of a clutch system to engage the gates.

Limitations and Problems:

- By modern standards, the rack and pinion operator is considered an antiquated means for gate operation.
- The operator may offer no mechanical advantage for operating the gate, or may include gearing to reduce the amount of input force required to operate a gate.
- Initially rack and pinion operators were manually operated by turning hand wheels or using lever bars. The gate operators may have been modified for operation by electric motors, and the speed of the motor often turned the operator faster than intended, providing little time for personnel to respond to a problem. In addition, the motor's output torque was often greater than the torque that could be put into the operator manually, hence motor-driven rack and pinion operators often have sheared teeth or racks.
- The rack and pinion operators were often used on powerhouse intake gates where the gate is normally raised or lowered under balanced hydrostatic heads. When used on intake gates, the gate operator did not always develop the force necessary to open or close a gate against unbalanced hydrostatic heads. This would require an alternate means to flood the downstream passages to balance the hydrostatic loads on the gate.
- The operator requires modifications to accommodate the use of electric power to operate.
- Normally the operator does not have the means or provisions to limit the input force used for operation. Therefore, the operator is often subject to damage and failure of rack and pinion teeth as a result of excessive input forces being applied.
- The pinions should be shielded or guarded to prevent debris from being lodged between the rack and pinion, potentially damaging teeth, and to prevent injury to operating personnel.
- Without observation, rack and pinion operators are not normally considered to be suitable for automation or operation and control from remote locations.
- Many operators were manufactured of cast iron and are susceptible to cracking and failure of the cast base plates and gearing.
- If the operator's base plate, or anchor bolts, or the bearings supporting the pinion shaft, are not tight or sound, the pinion will tend to ride up the rack (resulting in loss of driving force on the gate) whenever forces on the gate become greater than the driving force developed by the gate operator. Such action could damage the gate operator and endanger the safety of the operating personnel. Racking of a gate with twin stems can result in exacerbating the situation.
- Mechanical gears should be enclosed in a housing to prevent injury if the reduction gears were to fly apart if a free-falling gate were to stop suddenly. This is a common problem with rack and pinion gate operators especially those that are manually operated, or operated with a traveling donkey motor, when there may be more freedom between the main bull gear and the input pinion.

ii) Screw Stem Operator

A screw stem operator consists of a threaded stem generally fixed or pinned to a gate, with the stem raised, or lowered, by rotating worm or pinion gears that are driven by a gear reducer. A screw stem operator is capable of providing very large forces for raising or lowering a gate, with the magnitude of the force that can be developed being limited by the diameter and unbraced length (buckling strength) of the screw stem. The operator may be manually operated or motor driven. Electrically driven operators can be equipped with position indicators (mechanical or electronic), and in this case, the operator is suitable for operation via remote control. The screw stem operator is a simple technology that is also used to operate gates and valves in the water and wastewater industry.

Limitations and Problems:

- Screw stem operators can often transfer forces to, and damage, the supporting structure. Screw stem operators do not have a sound means of limiting the thrust or pull loads than can be developed. Motors can develop stall motor torque that may result in thrust or pull loads in the gate stems that are more than three to four times greater than the designed loads. The screw stem and supporting structure must be designed to handle the loads, or they may fail. While the operator is normally equipped with torque limit switches, the switches are intended only for the protection of the motor, not overload protection of the gate stem or support structure. To provide some means of limiting the maximum loads transferred to a support structure, additional mechanical torque limiting devices (external to the gate operator itself) can be added. Electrical devices (shunt breakers and relays) can also be added to the motor's power control circuitry to limit the current drawn by the operator's motor.
- Due to the slenderness of the screw stem (a function of the stem's length and diameter), stem guides may be required to prevent the stem buckling, and the stem guides themselves may be susceptible to damage and deterioration.
- Without electric power for operation, a manually operated hoist may not be suitable for use as an emergency closure gate, or to open a gate under emergency conditions. This is because it may not be possible to maintain the manual effort required to either open or close the gate, or to operate the gate in a timely fashion.
- Screw stem operators cannot balance the loads between the stems of a tandem-stem operator to insure a gate will lift and lower evenly.
- Stems need to be lubricated for proper operation, and the lubrication may be petroleum-based which could have environmental impacts. During severe cold weather, grease can stiffen causing the operator to stall due to the added load required to move the worm gear, and in hot climates a grease with a high melting point may be required. Non-petroleum-based lubricants are available, but the particular lubricant should be carefully selected, as not all are suitable for use in hot or freezing climates, or suitable for use if subjected to heavy washings by rain, or travel in water.

- Screw stem operators were often used to replace rack and pinion operators. Screw stem operators can be manufactured to develop significantly more lifting and thrust forces than could be developed with rack and pinion operators. Because of this additional capacity, a gate which was originally designed for a rack and pinion operator, is susceptible to being damaged due to the new ability to operate under loads for which the gate connections were not designed. Replacement screw stem operators were often sized to allow operation of a gate under unbalanced hydrostatic head, while the gate (especially wood) was designed to be operated only under balanced head conditions. If a screw stem operator is installed on an existing gate, attention must be given to the location and strength of the stem-to-gate connection. For wood gates, the stem-to-gate connection should be bolted to the bottom of the gate to allow the lifting of the gate from the bottom, rather than lifting the gate from the top timbers.

iii) Hydraulic Actuators

Hydraulic actuators may consist of a single or dual hydraulic cylinders operated by a single hydraulic power unit (HPU) to raise and lower a gate. The hydraulic actuators can be used in place of screw stem or rack and pinion operators on vertically traveling gates. Hydraulic actuators can also be used in place of hoists or winches, and to replace the wire ropes or chains with lifting stems or rods on radial-motorized gates (if the gate has been designed for operation by such means). For a facility with multiple gates, hydraulic actuators cost less to install than screw stem operators, because one HPU can be used to operate multiple hydraulic cylinders.

Hydraulic cylinders can be sized to provide very large thrust and pull forces, whose magnitudes can be controlled by pressure reducing valves. The hydraulic actuators can be designed with accumulators to provide the pressure needed to operate a gate without external electric power, making a hydraulic actuator suitable for use to operate an emergency closure gate, or to open a gate under emergency conditions. The hydraulic cylinder can be equipped with internal or external position indicators, providing a fine degree of control of the position of a gate. When the cylinders are equipped with position indicators, and the HPU with pressure relief or load limiting valves, a hydraulic actuator is a suitable operator for operation via remote control. The HPU can also be equipped with devices to balance the load between different cylinders, to insure a gate will move evenly.

Limitations and Problems:

- Oils used in the system may be petroleum-based and could have some environmental impact if spilled. When used in extreme sub-freezing locations the hydraulic oil is usually petroleum-based as used in aircraft, whereas in warmer regions, environmentally friendly “green oils” (food grade or biodegradable oil) may be used.

- Due to the slenderness of the cylinder rod, rod guides may be required to prevent rod buckling, and the stem guides themselves may be susceptible to damage and deterioration.
- Oil leakage from the cylinder seals, due to piston or rod seal failures, may be difficult to contain, and usually the cylinders are located over, or submerged in, a waterway.
- Hydraulic power systems have more components that require routine maintenance, and which may be susceptible to failure.

iv) Hoists and Winches

A hoist, or winch is a device that is used to raise or lower a gate, by wrapping a cable or chain, around or over, a take-up drum. A hoist requires power input to raise a gate, but lowers the gate by gravity with a hoist brake controlling the rate of descent. Hoists may be manually operated, but are generally driven by electric motors, although hydraulic motors can also be used. Hydraulic cylinders have also been used to ratchet the hoist's take-up drum.

A hoist can be equipped with position indicators to make it suitable for operation via remote control. For heavy lifts, the hoists can be designed with electronic equipment to balance the load between the two lifting points to insure that the gate will be lifted evenly.

Depending on the type and size of gate to be operated, a gate may have a dedicated hoist, or a mobile hoist or gantry crane, that may be used to lift multiple gates. Some facilities have a dedicated hoist for each gate, with a traveling donkey or mule motor to provide power to the hoist. When using mobile hoists, gantry cranes, or donkey motors, consideration should be given to the consequences of a mechanical failure of the mobile equipment. Ideally if there are a large number of gates to be operated, two or more mobile hoists, gantry cranes, or donkey motors should be used to provide redundancy.

Hoists may use cables (mild steel, stainless steel, or high strength steel), ordinary link chains (mild or high strength steel), or roller chains to connect the hoist to the gate. The connecting chains and cables may pass through sheaves attached to the gate, or a single chain or cable may be attached to the gate using a turnbuckle or other threaded device, thus allowing adjustments to the length of the cable and chains.

Limitations and Problems:

- Gates are closed only by gravity and the hoists are not able to apply a force to push a gate into the closed position. As a result, the weight of the gate must be great enough to overcome hydrostatic loads, down drag, and friction forces, which results in a heavier gate and an increased hoist capacity.

- Power as required for operation, although for vertical lift gates that are designed for emergency closure operation, a hoist may be suitable for use in an unpowered free-fall lowering of the gate.
- Hoists have significantly more moving parts than screw stem operators and hydraulic actuators, requiring more maintenance.
- Inspection of the lifting cables or chains may require specialized inspection and testing equipment, and experienced personnel.
- Because of their length and physical location, the chains, cables, and other normally submerged lifting components are difficult to inspect, it is difficult to maintain their protective coatings, and they require more maintenance.
- Roller chains, made from a series of parallel plates with pins and rollers are more difficult to inspect and maintain than are conventional cables and chains, but they can be designed to lift significantly greater loads. Keeping the movement of rollers and pins free is a maintenance concern and anti-seize products may be environmentally undesirable. Even under load the roller chain may not straighten out, changing the effective length of the chain and resulting in the gate not raising evenly, and manual hammering on the chain may be required to ensure that the chain straightens and properly engages the hoist's lifting teeth.
- A mobile gantry may utilize a lifting beam to raise and lower a large vertically operated gate such as radial or vertical lift gates and bulkheads used for emergency purposes. Problems that can occur with lifting beams include: floating debris blocking the gate's lift lugs; debris wedged or ice buildup in the beam's guide slots; ice buildup on the lift lugs; malfunction of the lifting beam sheaves or lift lug engagement device; concrete growth causing jamming of the beam in the guide slots, and inability to operate the gate(s) located beyond the mobile equipment that has broken down, or whose lifting beam is jammed in the guide slots.

d) Gate Vibration

Vibration is a frequent cause of problems with gates. Vibration can result in damage to the gate, the gate structure, or the operating equipment. Vibration may occur only at certain gate openings or under certain operating heads, or the vibration may have developed only after years of operations on an aging gate. The cause of vibration is often difficult to determine and assess, and exact conditions may not be repeatable as subtle changes in the conditions to which the gate is subjected may eliminate the vibration. If the hydroelectric project has a suitable number of gates, or does not have to rely on the operation of a gate in a specific position, then the simplest solution for eliminating the vibration is to not operate the gate in a position that results in the vibration.

Lewin (Lewin, 2001) identifies vibration as being classified into three types:

- Extraneously induced excitation that is caused by a pulsation in flow or pressure, which is not an intrinsic part of the vibrating system (the gate).

- Instability-induced excitation that is brought about by unstable flow. Examples are vortex shedding from the lip of a gate, and alternating shear layer reattachment underneath a gate.
- Movement-induced excitation of the vibrating structures. In this situation the flow will induce a force which tends to enhance the movement of the gate.

The problem of gate vibration is beyond the intent of the guidelines. For information on causes, assessment, and design to avoid vibration, the reader should consult Lewin (Lewin, 2001), Chapter 10. The text also has an extensive listing of references containing results of investigations and designs to address gate vibration.

6.2.3 Corrective Measures

The corrective measures associated with gates and gate hoists are described in conjunction with Section 6.2.2 – Problems.

6.2.4 Opportunities

Opportunities for upgrading of gates and hoists are often identifiable, and present themselves when work is being performed to resolve a problem, or when replacing a gate or hoist due to deterioration or age. Due to the longevity of a gate or gate hoist, they often remain in service for an extended period of time, possibly beyond that intended by the designer, eventually succumbing to obsolescence of function, operation, or safety. When opportunity presents itself, each gate and hoist system should be evaluated to determine if operations can be automated or otherwise modified to use limited personnel. The costs to upgrade a gate and hoist system should reduce the operational costs and safety risks to operating personnel.

The following are opportunities that may be considered for:

a) Gates.
b) Gate Seals.
c) Gate Hoist.

a) Gates

Opportunities for improvement on a particular type of gate are limited, mostly associated with improvements in materials, gate design, advancements in gate hoists, and gate controls. Because a hydroelectric project has so many different needs for flow release through the project, there is no single type of gate that can address all needs, and perform all functions. The single purpose of a gate, control of flow, is also the most opportunistic area for improvement, where a better control of flow releases can result in an increase in power generation or a reduction in operating costs. The post-1950s development of the rubber dam is an excellent example of a gate improvement to better control flow. The rubber dam provides a less expensive, adjustable, and controllable spillway gate to replace uncontrollable low head flashboards or numerous small gates located on a spillway crest. The development of the fuse gate in the 1990's enabled the raising of a dam without the need to construct large movable gates, yet still provide the necessary spillway capacity.

When investigating possible problems associated with gate movement, the reviewer should always be aware of any opportunities to improve the design of the gate to control the release of flows. Prior to implementing modifications or repairs, existing gates, guides, operators and sub-assemblies should be precisely dimensioned and photographed. Gates have often been modified or fabricated in a manner not reflected in the original construction documents, and these differences may be a cause of operating problems.

The advent of new materials or operating systems may not necessarily provide an opportunistic improvement in the design and operation of all types of gates found at a hydroelectric project. An example of this is the development of self-weathering steel (ASTM A588/A242). Gates fabricated of self-weathering steel may need to have a protective coating over the steel, as the force of discharging water and impact by debris, ice, and sediment tends to remove the protective rust, resulting in an accelerated rate of deterioration of the steel. Conversely, the continuing advances in protective coatings has resulted in the development of moisture cure coatings that can be applied in a damp or wet environment as is often found when performing repairs or maintenance in the field.

b) Gate Seals

Since the primary problem with gate seals is associated with leakage or reoccurring damage, both of which result in loss water, increased operating costs, and lost generation, replacing the seal with a better design will both correct the problem and result in an improvement to the seal's function. If there is little leakage or damage to a gate seal, there may be little economic or operational gain by replacing the seals with a better design.

When designing new or replacement seals, considerations should be given to the use of polytetra-florethylene (PTFE) clad seals with stainless steel rub plates, because the lower seal friction could result in the use of gate hoists with a lower rated lifting capacity. When designing gate seals, consideration should also be given to the means and methods to be used to replace the seals. Often the seals on the sides of a gate are inaccessible and cannot be replaced without the removal of the gate; and for seals that are accessible, there is often no means provided that will allow attachments to lift or reposition a seal for replacement.

c) Gate Hoist

Generally, the service life of a gate hoist may exceed that of the gate itself, if the hoist has been properly and routinely maintained. Unless a hoist has been operated beyond its rated capacity, upgrades or replacement of a hoist is often done as a matter of convenience for improvement of operation or control, or if an economic opportunity for replacement presents itself.

When replacing or designing a gate hoist, consideration must be given to the type and purpose for which the gate is being used, with the goal of providing a hoist with better operational control of the gate. There is no single hoist that is suitable for use on all gates for any application. Consideration should be given for a hoist that can be automated and be capable of operation without the need for tending or monitoring its operation.

6.2.5 Case Histories

No. 1 Poor Sealing and Gate Closure
Spier Falls Project (Brookfield Power New York, 1992)

The Spier Falls Hydroelectric Project in located in Gansevoort, NY, and the facility has a steel vertical lift gate (Broome or caterpillar) that is used as the powerhouse intake gate. The gate was constructed in 1930, and is 18 foot high by 13 foot wide, with a design head of 65 feet. The gate did not close properly or seal well because of badly pitted seal plates, and broken or missing roller chains and guides.

The gate was restored to proper operating conditions by the removal and replacement of the existing seal plates, roller chain guides and roller chains, with stronger and more corrosion resistant materials. Adjustments were also made to the existing bottom set screws on the gate to position and bring the gate seal surfaces in contact with the underwater sealing surfaces. The use of stronger and more corrosion resistant materials also reduced the friction forces acting on the gate and put less strain on the gate hoist.

No. 2 Tainter Gate Leakage, Freezing and Concrete Deterioration
Heuvelton Project (Brookfield Power New York, 1994)

The Heuvelton Hydroelectric Project, is located on the Oswegatchie River in Heuvelton, NY. The dam was constructed in the 1920's and included five 27 foot long by 10 foot high tainter gates, and one 27 foot long needle beam section. The gates were used for flood and reservoir level control, and were lifted independently with dedicated, electrically operated chain hoists.

During winter months the tainter gates would ice up severely due to seal leakage and precipitation. Every time an adjustment in the gate opening was required, significant effort was required to remove the ice to allow the gate to operate. When the site was tended 24-hours a day, ice removal was less of a problem, as plant personnel would continually work to minimize ice accumulation. Once the plant went into unmanned operation the labor costs increased dramatically for ice removal, which was then performed primarily when a gate had to be operated. In addition, the concrete structure that contained the gates had deteriorated and in need of replacement.

The owner of the project wanted to eliminate or greatly reduce de-icing operations at this facility whenever the gates needed to be operated. They also wanted to start a program to restore the deteriorated condition of the gate structure. Discharge calculations indicated that two of the gates were needed to discharge normal winter flows, which identified how many tainter gates and associated portion of the gate structure needed to be rebuilt. The remaining three gates were to remain in their current condition and method of operation.

The owner evaluated the costs to remove and rebuild two gates and gate operators, including the addition of heaters and ice shields to minimize the icing problems. Because of the need to also rebuild the concrete structure, the owner evaluated the economics of installing two rubber dams in place of the tainter gates. By the nature its design and manufacture, the rubber dam itself is not considered to be susceptible to freezing conditions, even if overtopped by wave action. However, the rubber dam's intake, exhaust, and drain lines can freeze due to moisture accumulation and failure to periodically drain the lines. Based on economic and operational considerations, rubber dams were installed in two of the gate bays.

Installing the rubber dam equipment and control system provided an opportunity to equip the site with an automatic pond level control, further reducing the operation requirements and costs for personnel to visit the site to make adjustments to the gates. During periods of high flows, the remaining tainter gates are adjusted as needed to provide a "coarse" control of the spillway discharge, and the rubber dams provide the automated fine control to maintain the level of the impoundment for optimum energy production and flood control.

No. 3 Trunnion Pin Failure
The Dalles Project (USACE, 2001)

The Dalles Project is located in The Dalles, OR, and became operational in 1957. The Project has twenty-three 50 foot wide by 48 foot high tainter type spillway gates, in which the existing trunnion design utilized a greased bronze bushing mating to a cast steel pin. The bushing was manually greased from a remote location via long grease lines.

One of the spillway gates was damaged when the trunnion pin rotated and sheared off one of the keeper plate bolts. The problem developed when the lubricating grease did not reach the bushing and pin, resulting in the development of high friction loads in the bearing.

Options evaluated to repair the gate included: replacing the bushing and lubricating system in kind but with stronger bolts for the keeper plate; replacing, as previously described, but using an automated mechanical lubrication system; or, as ultimately selected, replacing the bushings with bushings manufactured from a self-lubricated material, rotating on new stainless steel trunnion pins.

By using self-lubricated bushings, the trunnions no longer need to be greased which has eliminated the periodic maintenance requirement, and also, the potential for petroleum contamination of the adjacent waterway. The self-lubricated bushings also have a lower friction factor and result in less operating load of the gate hoist. Although the repairs involved the rehabilitation of the trunnions at only one of the existing spillway gates, the knowledge gained from the failure investigation, the modifications made, and the efforts required to repair the trunnions provided the owner with first hand information of requirements and costs for the possible upgrading of the trunnions for all of the other tainter gates.

No. 4 No Provisions for Maintenance Bulkheads
Reusens Project (AEP, 1998)

The Reusens Hydroelectric Project is located in Lynchburg, VA, and the gate portion of the project was constructed in 1931. The dam's spillway contains eight 44 foot wide by 16 foot high vertical lift wheel gates, operated by individual cable hoists on a bridge structure located over the gates.

After years of service, the spillway gates were in need of general repair and repainting. The repairs would be best performed with the gate in the raised position, but the gate structure was not designed to accommodate the installation of maintenance bulkheads, and to repair the gates in the dry would have required lowering the head pond for the duration of the repair. Lowering the head pond would not only have impacted on the generation of power from the hydroelectric project, but also on the local municipality water supply intake upstream of the project. Lowering the impoundment for an extended period of time was not an option.

To facilitate the repairs, it was decided to install a maintenance bulkhead. Two bulkhead options were evaluated. The first required the addition of a monorail system to the existing steel superstructures to transport the bulkheads in sections, and position and lower them into new slots cut into the concrete gate piers. The second option was to use a hinged-section, floating bulkhead which would fit into a vertical seat cut in the upstream nosing of the gate piers. The floating bulkhead system was selected, based on an economic evaluation. A single bulkhead, 46 feet wide by 20 feet high, was made up of five-4 feet deep box sections, which are pinned together and floated into place. The piers required a minimal amount of modification to accommodate the bulkhead. When the bottom chambers of the bulkhead are flooded the bulkhead rolls down the upstream face of the gate structure, and into place like a garage door.

Maintenance can now be done to the spillway gates when needed and without losing generation by having to lower the pond. The bulkheads can be floated into place and installed in less than four hours, and with minor structural modifications, it may be possible to use the bulkheads at the owner's other projects nearby.

No. 5 Excessive Movement and Tainter Gate Failure Elkhart Project (AEP, 1997)

The Elkhart Hydroelectric Project is located in Elkhart, IN, and was constructed in 1913. The dam's spillway contains eleven 25 foot wide by 10 foot high tainter gates which are operated by a traveling chain hoist.

During normal operation, one of the gate arms failed due to increased side friction against the concrete piers, caused by excessive movement of the gate. The problem occurred when the gate was being raised and became jammed, although plant personnel where able to close the gate by jacking it shut.

Repairs were considered minimal, and it was decided to repair the gate in place by installing a maintenance bulkhead upstream of the gate, to allow the work to be performed in the dry. The repairs consisted of replacing both trunnion arms and tie rods and realigning the gate. The lower channel on the gate, which contacts the sill plate to help keep the gate from racking, was also adjusted. The cause of the problem was that when side heaters were installed in a post-construction modification made to the gate adjustments had been improperly made and it was slightly out of alignment. Over the course of the years the misalignment worsened and the trunnion arm eventually buckled. The trunnion bearings were not replaced as they were not damaged.

No. 6 Insufficient Gate Capacity and Loss of Generation Byllesby Project (AEP, 1997)

The Byllesby Hydroelectric Project is located in Byllesby, VA, and was constructed in 1912. The dam's spillway consists of six 30 foot wide by 8.5 foot high tainter gates and fifteen 33 foot wide by 9 foot high stanchion stoplog type flashboards. Stanchion stoplogs consist of closely spaced vertical beams with wood stoplogs spanning between the beams. Flow is discharged by manually unlatching the beams allowing them to drop and release the stoplogs.

During periods of high flows that exceeded the capacity of the spillway gates and the turbine generating units, a portion of the stoplogs may become overstressed, and fail, due to the elevated level of the impoundment, or some of the stanchions may be unlatched to provide additional discharge capacity. To reset the stoplogs required waiting until the flow in the river had reduced to a level that could be discharged through the turbines, and then lowering the impoundment to the base of the stoplogs to allow manual resetting of the stanchions and installation of new stoplogs. To reduce the cost of lost generation due to the loss of the stoplogs, and reduce the cost of labor and material for the replacement of the stoplogs several times a year, it was decided to add one additional spillway gate to increase the spillway discharge capacity before the stoplogs would fail, or required releasing.

Options evaluated to replace a section of stanchion stoplogs included the installation of an additional tainter gate, installation of a vertical lift gate, and the installation of a bottom hinged crest gate. Based on an economic evaluation, the least cost option that was selected was the installation of an inflatable rubber dam that operated from compressed air. The rubber dam was designed to fit into the opening between the existing concrete piers with minimal modifications to the pier or existing concrete crest.

The installation of the rubber dam did result in significant savings in operational costs (labor and materials), and an increase in power generation, because the head pond could be maintained at normal operating levels under conditions of higher flow. Following those flow events that would have originally required the release of the stoplogs, the lowered rubber dam could be raised without need to have the impoundment lowered, which also contributed to the increase in power generation.

No. 7 Uncontrolled Flashboard Failures
Allens Falls Project (Brookfield Power New York, 1991)

The Allens Falls project is located in Parishville, NY, and was constructed in 1927. The project's dam included a 550 foot long concrete Ambursen spillway, containing 2 foot high wooden flashboards, and no other significant means to control the discharge of flows over the spillway.

The level of the Allens Falls impoundment fluctuated by a couple of feet throughout the year because sections of the flashboards would fail during flood flows or from ice pressure, with the result being a lower pond until the flashboards could be repaired. The fluctuation reportedly affected fish habitat and fish reproduction.

To eliminate the uncontrolled fluctuation of the impoundment, the flashboards either had to be protected from failing or removed. The options that were investigated included: installing a bubbler system along the full length of the spillway to minimize failure of the flashboards from ice pressure; permanently raising the spillway crest by 2 feet, eliminating the flashboards and installing a controllable mechanical means of discharging flow over the dam, in order to maintain the same flood passage capability; or removing the flashboards and reduce the normal pond elevation by 2 feet. The bubbler system was not selected because it would not prevent failure of the flashboards during floods or from debris impact, and a lower pond would adversely affect plant generation. The option selected was to permanently raise the spillway crest by 2 feet using reinforced concrete, and to add two large steel crest gates. Each steel crest gate is 9 foot high by 60 foot long with a bottom hinge, and is raised and lowered using hydraulic cylinders. Steel crest gates were chosen over inflatable rubber dams because they were less likely to be damaged by vandalism or stray gunshots during hunting season.

The mechanical crest gates not only eliminated the fluctuation of the impoundment, but also resulted in the ability to maintain a higher impoundment level year-round, and therefore increased hydroelectric power generation. The mechanical gates also eliminated the material and labor costs, and danger and liability associated with replacing and maintaining the flashboards.

No. 8 Safety Concerns with Antiquated Gate Operator
Belfort Hydroelectric (Brookfield Power New York, 1997)

The Belfort Hydroelectric project is located in Croghan, NY, and was constructed in 1903. One of the penstocks had a single 10 foot high by 7.5 foot wide wooded headgate, with a manually operated rack and pinion gate operator.

The existing rack and pinion had broken gear teeth, and was physically demanding and dangerous to use. The gate operator had exceeded its useful service life.

The gate was used as a maintenance gate for the dewatering of the penstock and turbine. Because the headgate was not needed to control the flow of water into the penstock, there was no need to provide a mechanical gate operator with the ability for remote or untended operation. Due to the simple needs for the gate operator, an inexpensive cantilevered steel lifting frame was built and supported from the intake deck, and an electric hoist that could raise and lower the headgate was suspended from the frame. The lifting frame also enabled the gate to be lifted free of the gate slots for future maintenance.

No. 9 Sector Gate Rehabilitation
Post Falls Hydroelectric Development (MWH, 2004)

The Post Falls Hydroelectric Development is an 18 MW hydropower project constructed in 1906 on three channels of the Spokane River in Post Falls, ID. The project consists of a concrete gravity dam and integral powerhouse on the Middle channel; a concrete gravity dam overflow spillway with six sluice gates on the South channel; and a 430-ft. long concrete gravity dam spillway with eight radial gates and one large rolling sector gate on the North Channel (Figure 6.2-4). Over 40% of the dam's spillway capacity is supplied by the 100-ft. wide bay of the North channel spillway that is controlled by the rolling sector gate. The sector gate is 100 feet long by 14 feet high and was constructed in 1922 to replace a timber bear-trap gate that was part of the original construction.

The rolling sector gate is a rather unique gate, having qualities of both a radial gate and a roller dam gate. The gate functions in a manner similar to a radial gate, but instead of pivoting about a single axis (trunnion), the ends of the gates have large diaphragms with geared perimeters that engage pier-mounted racks on either pier. As the gate is raised with a 42-ton chain hoist, the gate "rolls" up these tracks.

Figure 6.2-4 Post Falls Dam – Sector Gate Spillway (courtesy of MWH)

Due to the age of the rolling sector gate, and a change in the loading conditions that resulted from a previous 1.5-ft. pool raise, a condition assessment and structural analysis was performed in 2001. It concluded that the gate needed additional structural reinforcement to bring the gate up to modern standards. In addition, the gate required general rehabilitation, including painting and seal replacement, and the hoist required reconditioning. To make the required structural modifications, the gate would have to be unloaded.

An underwater inspection identified heavy deterioration of portions of the spillway concrete that was placed in the early 1920's when the spillway bay was converted from a bear-trap gate bay to a sector gate bay. The installation of the sector gate required encasing the bear-trap gate conduits/alcoves and the addition of concrete pier extensions. Portions of this concrete, placed underwater during construction, had since eroded away, leaving large voids underwater that required repair.

Maintenance of the sector gate had historically been difficult, because no means of dewatering the spillway bay was ever provided, and the impoundment is never lowered enough to enable gate maintenance work to be performed in the dry. In the past, when maintenance to the upstream portion of the gate was required, the gate was rolled up 90 degrees until out of the racks. The upstream gate skinplate was then facing directly up and work could be performed on the skinplate and seals by walking across the skinplate. This however, had to be done while water was spilling beneath the gate.

It was decided that the gate would have to be dewatered to perform the rehabilitation. Lowering the impoundment to the sill level was not an option because it would require lowering the lake 10 feet below its normal minimum pool level, which was unacceptable and impractical for many reasons. Performing modifications while spilling was also not practical for safety reasons and because of severe logistical implications for construction.

In order to perform the rehabilitation work, a 20 foot high, multi-bay stoplog system was constructed to dewater the gate bay. The dewatering system utilized the vertical face of the spillway sill and a 114 foot long steel truss (that spans the width of the spillway at the top) to support a series of timber stoplogs (Figure 6.2-5). This system was relatively simple to install, required the least amount of underwater work, and had the lowest expected cost of the alternatives studied.

The 114 foot long top support truss of the dewatering system was essentially a two dimensional truss, orientated in a horizontal plane. The top support truss, concrete sill, 7 intermediate vertical guide columns and 2 end pier-mounted brackets framed the 100 foot wide spillway opening into eight, 12.5 foot wide bays for support of timber stoplogs. The vertical columns were connected to the downstream chord of the truss at the top and rested (under load) against the vertical face of the sill at the bottom. The bottoms of the columns were retained in adjustable, sill-mounted stop brackets. Two end brackets were temporarily mounted to the vertical face of each pier to anchor the truss. Knee braces extend off several of the vertical columns to support the upstream side of the truss against its self-weight. The timber stoplogs, bundled into 12.5 foot wide by 10 foot high sections with tie rods, formed buoyant panels which were placed, two high, into each bay using the vertical columns as guides and supports. A heavily ballasted lifting beam pushed the panels down into position, at which time they were fixed to the columns. The design of the structure provided ample adjustability to facilitate convenient installation in the field. Special connections, sliding pads, oversized holes and elastomeric bearings were used to accommodate deflection and misalignments of the structure. A bottom rubber J-seal and ample amounts of sawdust pellets resulted in a nearly leak-tight system.

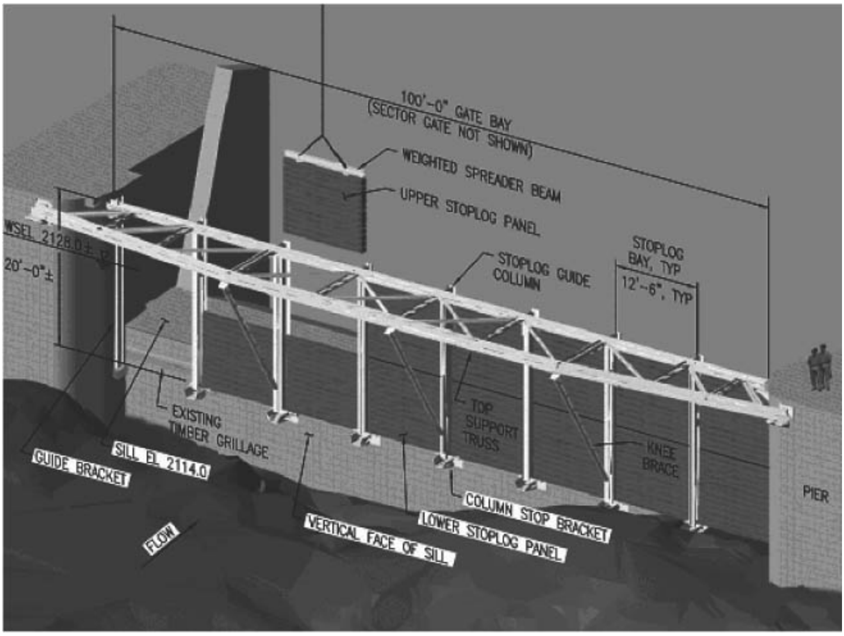


Figure 6.2-5 Post Falls Spillway Dewatering System (courtesy of MWH)

Prior to installation of the dewatering system, underwater civil modifications had to be made to repair the damaged concrete that could not be dewatered. Localized repairs were also needed to provide a sealing surface for the dewatering system. Steel formwork was installed, reinforcement set, and tremie concrete placed at the piers to restore them to their original shape.

Concerns over the ability of the concrete sill to support hydrostatic uplift of the dewatered spillway bay required additional underwater work, once construction had already begun. As part of the 1922 modifications, a 2.5 foot thick slab of concrete was added on top of the original concrete sill (Figure 6.2-6). A continuous timber horizontal grillage, approximately 16 inches high, was sandwiched between the original concrete and the 2.5 foot thick slab. The presence of this grillage was observed during the original underwater inspection, and the dewatering structure was designed to be set and sealed below it to prevent uplift on the slab after dewatering. After further cleaning of the underwater concrete, during construction, a horizontal construction joint was found approximately 1 foot below the timber grillage. This construction joint ran the width of the spillway and raised concerns that uplift could develop at this joint. To avoid potential collapse of the sill slab, twenty-six 1 inch diameter, 10 foot long, epoxy rock anchors were installed underwater across the width of the spillway, to tie the upper slab to the original mass concrete of the spillway.

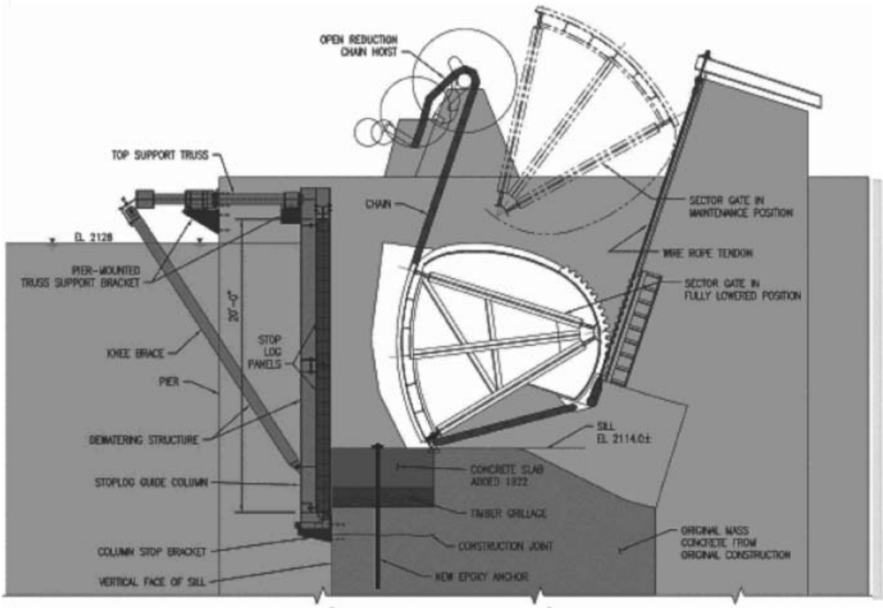


Figure 6.2-6 Cross Section of Sector Gate, Civil Structure and Dewatering System (courtesy of MWH)

Epoxy anchors were used in lieu of mechanical anchors due to concerns over the quality of the original concrete to take high point loads from the anchors. Divers cored holes underwater through the concrete slab, timber grillage and 6-ft. into the original spillway monolith using a custom jig. The anchors were set and the specified amount of epoxy placed into each hole. Pull testing of all anchors up to their design load was performed. Twelve of the anchors failed the test. A post failure investigation of the anchors concluded that the epoxy grout had seeped out of the holes through apparent voids and seams in the original concrete, leaving only a small fraction of epoxy to support the load. The grouting process was repeated and the anchors were successfully re-tested.

After dewatering, the following modifications were made to the gate:

- Undersized structural members were replaced with higher capacity structural members and slip-critical bolted connections replaced riveted connections.
- The gate was then repainted using a high solids epoxy paint system with an aluminum-flake epoxy prime coat. Prior to repainting, the original hazardous lead paint system was removed by sandblasting, necessitating the construction of a containment structure, composed of shrink wrapping over a scaffold framework, around the gate.

- The deteriorated timber bottom and side sealing system was replaced with an Ultra-high Molecular Weight Polymer (UHMWP) bearing/rubber seal system.
- The 2 inch diameter steel wire rope tendons, used to support the gate when it is rolled out of the rack for maintenance, were replaced in kind.
- The carbon steel sill beam was replaced with a new stainless steel sill beam assembly, grouted into place to the existing sill.
- Heavily pitted carbon steel embedded side sealing surfaces were restored using a two component, ceramic epoxy repair compound and repainted.

The following modifications were made to the gate hoist as part of the rehabilitation:

- Two existing, 22-ton capacity link type hoist chains were replaced with new hoist chains, equipped with grease ports and grooves in every pin for convenient lubrication.
- Worn and damaged chain sprockets were replaced with new hardened steel sprockets.
- Worn gears and pinions in the open gearing system were replaced with new hardened gear and pinion combinations.
- In kind replacement of the hoist solenoid holding brake.
- Replacement of worn babbitt shaft bearings with new self-lubricating composite bearings.
- General cleaning and painting of existing equipment.
- Demolition of hoist houses and replacement with new timber framed buildings.

The $1.5 million rehabilitation project was performed over an 8-month period, including a 5-month gate outage. The following key experiences came about from the project:

- Rehabilitation of aged structures and equipment often brings many surprises. Construction budgets should anticipate this because it is unavoidable that extra work orders will arise once construction is begun. In the case of this project, approximately $500,000 worth of extra work orders were needed to cover additional work items exposed during the rehabilitation. Among these changes were: the need for replacement of various worn hoist components, which were inaccessible until the hoist was disassembled; and the design and installation of the tie-down anchor system that was determined necessary after further underwater investigation took place during construction.
- There are often discrepancies between original drawings and actual field conditions. Critical dimensions must be verified prior to initiating fabrication. If this cannot be reasonably performed during the design phase, them the contractor should be required to verify at the start of the project. In the case of this project, responsibility was placed on the contractor to perform an underwater inspection and verify field dimensions immediately after initiation of the project, and before finalizing fabrication design. This proved valuable many times during construction. There were also instances where this was not done which resulted in field modifications.

- The underwater work was clearly defined in the contract and an underwater inspection videotape was supplied to prospective bidders to assist them in estimating costs for underwater work. This proved to be a valuable asset during the bidding phase and helped eliminate underwater surprises to the Contractor.
- When underwater work is involved, an attempt should always be made to perform as detailed an inspection up front as possible, to minimize extra work orders later on. Repairs required after discovery of the hidden construction joint resulted in an almost 3-week delay in construction.
- The designer should be involved in the QA/QC process during fabrication and installation. The designer can often identify deviations from the design in the fabrication and can best judge the need for correction because of his/her intimate familiarity with the design.
- The dewatering system worked extremely well. The adjustability built into the design of the dewatering system components simplified installation. This feature was especially important, because both underwater and above water installations had to work together. After mounting the various support brackets that were needed for the structure, the dewatering system was installed over a 5-day period and was removed in 2 days.
- Project background data was invaluable during the design phase and was worth the significant effort, on the part of the owner, to retrieve the available data from the company archives. Ample information was available from the original construction, mainly photographs, that provided many hints and clues to the original construction methods and potential problem spots.

6.2.6 Collective Knowledge

a) General Information and Design

1. American Society of Civil Engineers (ASCE). (1989). *Civil Engineering Guidelines for Planning and Designing Hydroelectric Developments – Volume 2, Waterways.* ASCE, New York, NY.

2. American Society of Civil Engineers (ASCE). (1995). *Guidelines for Design of Intakes for Hydroelectric Plants.* ASCE, New York, NY.

3. American Society of Mechanical Engineers(ASME). (1996). *The Guide to Hydropower Mechanical Design - Chapter 7.* HCI Publications, Kansas City, MO.

4. Creager, W.P. & Justin, J.P. (1950). 2nd Edition. *Hydro-Electric Handbook.* John Wiley and Sons, New York, NY.

5. Lewin, J. (2001). *Hydraulic Gates and Valves.* Thomas Telford, London.

6. Sagar, B.T. (1995). *ASCE Hydrogates Task Committee Design Guidelines for High-Head Gates.* ASCE Journal of Hydraulic Engineering, Vol. 121, No. 12, December, 1995, Paper No. 5480.

7. Thang, N. and Naudascher, E. (1993). *Approach-Flow Effects on Downpull of Gates,* ASCE Journal of Hydraulic Engineering, Vol. 109, No. 11, November, 1983, Paper No. 18357.

8. United States Army Corps of Engineers (USACE). (1994). *Mechanical and Electrical Design of Pumping Stations.* [EM-1110-2-3105]. United States Army Corps of Engineers. (Updated 1999).

   http://www.usace.army.mil/publications/eng-manuals/em1110-2-3105/toc.htm.

9. United States Army Corps of Engineers (USACE). (1995). *Hydroelectric Power Plants Mechanical Design.* [EM-1110-2-4205]. United States Army Corps of Engineers, Hyattsville, MD.

   http://www.usace.army.mil/publications/eng-manuals/em1110-2-4205/toc.htm.

10. USSD. (2002). *Improving Reliability of Spillway Gates.* United States Society on Dams, Denver, CO.

11. Zipparo, V. & Hasen, H. (eds.). (1993). 4th Edition. *Davis' Handbook of Applied Hydraulics.* McGraw-Hill, New York, NY.

At the time of the preparation of these Guidelines for upgrading or extending the life of civil works of a hydroelectric project, the following documents were also being prepared:

10. American Society of Civil Engineers (ASCE) Hydropower Task Committee was preparing guidelines with the working title: *Condition Assessment of Water Control Gates.*

References 3 and 5 provide a very good overall review of selection criteria, design and operational parameters for gates. Reference 8 provides good detail on the structural, mechanical, and electrical considerations of spillway gate operation, including an excellent section on design practice and materials selection.

b) Design Standards

Standards for gate design are limited. Prior to 1930, there were few standards in general use for the design of gates, as most gate design standards were developed by the design firm or the engineer of record. Federal agencies such as the Army Corps of Engineers and the Bureau of Reclamation have developed standards of their own. As of the date of these Guidelines, gate design standards are still limited, with designers and manufacturers both still using in-house standards, in combination with standards published by the following agencies:

11. American Institute for Steel Construction, Chicago, IL. http://www.aisc.org/

12. American Welding Society, Miami, FL. http://www.aws.org/

13. German Institute for Standardization (DIN). (1998). *Hydraulic Steel Structures – Criteria for Design and Calculations.* [DIN 19704].

14. German Institute for Standardization (DIN). (1998). *Hydraulic Steel Structures - Recommendations for the Design, Construction and Erection.* [DIN 19705].

15. German Institute for Standardization (DIN). (n.d.) *Hydraulic Steel Structures - Water Control Structures I.* [Handbook 179].

16. Japanese Technical Standards for Gates and Penstocks.

17. United States Army Corps of Engineers. *Design of Hydraulic Steel Structures.* [EM-1110-2-3105]. http://www.usace.army.mil/publications/eng-manuals/em1110-2-3105/toc.htm

18. United States Army Corps of Engineers. *Vertical Lift Gates.* [EM-1110-2-2701]. http://www.usace.army.mil/publications/eng-manuals/em1110-2-2701/toc.htm.

19. United States Army Corps of Engineers. *Design of Spillway Tainter Gates.* [EM-1110-2-2702]. http://www.usace.army.mil/publications/eng-manuals/em1110-2-2702/toc.htm.

20. United States Army Corps of Engineers. *Wire Rope Selection Criteria For Gate-Operating Devices.* [EM-1110-2-3200]. http://www.usace.army.mil/publications/eng-manuals/em1110-2-3200/toc.htm.

6.2.7 Technical References

American Electric Power (AEP). (1998). Project files for American Electric Power, Columbus, OH.

American Electric Power (AEP). (1997). Project files, American Electric Power, Columbus, OH.

Brookfield Power New York. (1997). Project files, Brookfield Power New York, Liverpool, NY.

Brookfield Power New York. (1994). Project files for Brookfield Power New York, Liverpool, NY.

Brookfield Power New York. (1992). Project files for Brookfield Power New York, Liverpool, NY.

Brookfield Power New York. (1991). Project files, Brookfield Power New York, Liverpool, NY.

Lewin, J. (2001). *Hydraulic Gates and Valves*, Thomas Telford, London.

MWH. (2004). Project files for Avista Utilities, Spokane, WA.

United States Army Corps of Engineers (USACE). (2001). Project files for USACE, Portland District, Portland, OR.

United States Society on Dams (USSD). (2002). *Improving Reliability of Spillway Gates*. USSD, Denver, CO.

6.3 Valves and Operators

6.3.1 Function

Valves are mechanical devices used to control flow and pressure in pressure conduits, or as emergency closure devices to protect sections of pressure conduits as well as malfunctioning equipment in the powerhouse. Valves may be located anywhere there are pressure conduits in the dam, spillway, or in an outlet structure. Valves may be used for protection and isolation of sections of penstocks, and as guards to allow access for maintenance or repairs of downstream devices, that may include the turbine-generating equipment or an energy dissipating valve. Valves perform many critical duties such as controlling flow and pressure through and around hydroelectric plants and dams, and the release of air and vacuum in pressurized water conduits.

The difference between a gate and a valve has to do primarily with their location. Valves are located in, or at the end of a pipeline (pressurized conduit), whereas gates are located in a freestanding structure rather than inline with a pipe. Other differences between a gate and a valve have to do with their construction and the means of seating and support of the gate or valve. Valves are provided with bodies that incorporate seating surfaces to transfer load from the disc to the body, and the interior of the valve is pressurized. Gates are not provided with bodies, but with separate guides, rails, and sealing surfaces that are attached to an independent structure, and the hydrostatic pressures are external to the gate.

Both valves and gates are fabricated assemblies, with the design of a valve being based on allowable deflections and stresses, and the design of gates being based on allowable stresses. A valve requires sealing, seating, and mating surfaces that have been machined to a fine level of tolerance. Fine machining is not considered to be common with most fabricated gates as flexible seals are used to account for lower fit tolerances. The stiffness of valve discs and bodies relative to distortions are generally higher than for a gate in order to accommodate the machined mating surfaces and to minimize leakage. Since gates are generally much larger in surface area than valves, material cost savings are realized by incorporating less stiffness. Because of the stiffness of the valve, and the machining tolerances to which they are fabricated, valves are used in higher head applications than gates although the size of the valve often diminishes with increasing head.

There are a number of different types of valves that are commonly found in a hydroelectric project, and their primary function can be divided into three categories, that of Isolation, Control, or Discharge. The definition of the categories is as follows:

- Isolation: Valve used as a closure device under normal (either open or closed) operating conditions, or as a guard valve for emergency closure or maintenance purposes. With few exceptions, isolation valves are not suitable for flow control. Valve is located in-line and often has no backup closure device, or the backup device may be a gate located in an intake structure.

- Control: Valve with low energy dissipating qualities that can be adjusted to control flow with minimal headloss. Valve is located in-line and usually has a isolation valve as backup or guard valve.
- Discharge: An adjustable valve use to control flow and dissipate energy by air friction and entrainment. Valve may be located in-line or at the terminus of the line. Usually has a isolation valve or guard valve as backup.

A listing of the various types of valves and their application as may be found and used at a hydroelectric project, are shown in Table C-2 of Appendix C. Some of the valve types used on hydroelectric projects, although large in size, are not unique in application. Smaller valves are common on mechanical piping systems used to operate mechanical equipment, and can be found in an industrial plant or home. Valves, such as the butterfly, gate, knife, globe, ball, plug, and needle, perform the same functions as those controlling flow at a hydroelectric plant with the only difference being their smaller size.

Some hydroelectric turbines such as Francis, Kaplan, and bulb, often contain a valve, known as wicket gates, that are used as the primary means to control flow to the turbine. Wicket gates are not gates in the sense of the types of gates used on a dam, nor are they truly a valve. Turbine wicket gates are often identified as adjustable guide vanes that have seating and supports like those of a valve and are located in a pressurized conduit. With respect to the turbine's shaft, the wickets gates are arranged radially around the turbine runner, and consist of a number of overlapping wickets (rectangular discs) that operate in a similar manner to that of a round butterfly valve disc. In the context of this guideline, wicket gates are not considered to be a valve.

6.3.2 Problems, Causes, Solutions

This subsection of the guideline differs slightly in format from other similar sections in that discussions of problems, causes, and solutions are presented together.

The service life of a valve is dependent on: the type of valve; the use and frequency of use of the valve; the severity of the conditions under which the valve operates; operating the valve within its design parameters, and routine maintenance. Butterfly or spherical valves that are operated infrequently as guard valves, may have a service life of up to 100 years. An energy dissipating valve, such as sleeve valves, may have a service life of only 25 years when operated very frequently and under severe, abrasive conditions.

Valves of modern design (post-1950) generally have an expected service life in excess of 75 years if subjected to proper and routine maintenance. Wearing of seals or bearings, which was a serious maintenance problem for pre-1950s valves, has been mitigated through the development of better corrosion and wear resistant materials. However, even modern values that are infrequently operated will have a greater occurrence and frequency of problems. In addition, some valves (butterfly and spherical) can be manufactured with a second maintenance seal that would be engaged only when work is being performed on the main seal. With the use of computers to model stresses in the valve, they have become lighter, with thinner walls, resulting in ultimate factors of safety that are not as high as with valves fabricated before the 1970's.

A consistent goal in the design of valves has been to allow the valve to be operated and controlled remotely, without the need for personnel to tend and monitor the valve's movements. Post-1950 improvements in electronics, instrumentation, and the advent of the computer, have provided the means to integrate and totally automate the operation and control of a valve.

Table C-2 in Appendix C lists the key advantages and disadvantages of a particular type of valve. Some of these disadvantages translate directly into problems common to that type of valve. The problems, causes, and solutions presented in the sections that follow identify problems considered to be representative and common to all valves, regardless of type, and some uncommon problems that can occur with a particular type of valve. Many problems with a valve, other than design-related, are the result of valve function, and conditions to which the valve is subjected, including operation and maintenance. The problems described are not listed in any particular order of importance, or frequency of occurrence. The solutions presented are intended to extend the service life of the valve and operator system. Problems associated with valves are divided into two sections:

a) Common Problems.
b) Valve Specific Problems.

a) Common Problems

Valves, while not a principal capital component of a major project or plant, nonetheless deserve careful attention. Valves can be considered as analogous with brakes in an automobile: it is a problem if the engine does not start, but it is life threatening if the brakes do not work when needed. Because of the important role valves perform, the valve must be suitable for the intended application, properly designed and properly operated and maintained. Valves, and their operators, must function correctly if they are not to cause problems, or even a catastrophic failure that could lead to property damage, loss of revenues or water, and loss of life.

Common problems presented in the following sections include:

i. Application.
ii. Design.
  - Hydraulic Performance.
  - Mechanical Performance.
iii. Seats and Seals.
iv. Bearings.
v. Corrosion/Erosion/Water Quality.

vi. Operators.
vii. Valves with Self-Closing Tendencies.
viii. Large Valves and Deflection.

Causes of common problems associated with all valves are listed in Table C-3 of Appendix C. It is important to understand the cause of common valve problems so that proper corrective action can be taken to avoid future problems. While lubrication, adjustments, painting, and looking for loose parts are appropriate practice for maintaining all equipment, including valves, many problems are uniquely associated with particular types, classes, and styles of valves (see Table C-2, Disadvantages).

The remainder of this section discusses problems common to all valves along with possible solutions, followed by problems and solutions common to a particular type of valve:

i) Application

It is most important to properly select the most suitable valve for the intended application, and ensure it is designed and sized properly. The operation and life of a valve will be affected by: pressure, flow rates, open and close time, temperature, entrainment of debris and foreign objects, and weather. The latter can impact the valve operator, and sub-freezing conditions will impact the operation of many types of valves. Therefore, all of the above aspects must be considered when specifying and designing and also when analyzing problems with valves.

A valve's design pressure is very important, but it does not mean that a valve is necessarily suitable for high flow rates, or quick shutoffs, as may occur in emergencies, or even opening and closing under flow, as these conditions often introduce extreme conditions for which a valve may not have been intended. Often, the valve installed at project construction was the best device then available (typical of plants constructed pre-1940), whereas, by current standards, that particular type of valve would not be considered as the best choice for the intended application.

Solution:

The only true solution for a valve that is not suited for its present operation is replacement. While it may be possible to modify the conditions under which the valve is operated, such modifications are a compromise, and it is possible that such compromises may end in a catastrophic situation.

ii) Design

Hydraulic Performance

The design of a valve should be based upon laboratory hydraulic model tests to analyze the headloss, flow capacity, and cavitation characteristics of a particular hydraulic shape, as each valve, when shaped differently, reacts differently. Even a properly designed valve may not perform as well under field conditions as it did in the laboratory, simply because field changes made during construction, or the parameters under which the value is operated, may not have been modeled when the valve was designed.

Valves are designed to operate under pressure, and while normal design pressures are easy to predict, unexpected conditions from pressure raises, hydraulic column separation, or pulling vacuums, may present problems. Under flowing or leaking conditions, cavitation is often a major problem with valves. Only the multi-jet sleeve-type valve is designed to withstand continuous cavitation conditions. Prolonged cavitation in most valves should usually be avoided, as it will eventually erode, vibrate, and weaken a valve.

Solution:

The only true solution for a valve that is poorly designed or not suited for its present operation is replacement. Even valves that were properly designed and ideally suited for the intended operation, were not designed to operate with optimum efficiency over an unlimited range of operating conditions. When a valve is not operated within its optimum efficiency or range of conditions, it tends to cavitate or vibrate. If cavitation or vibration occurs only at certain operating heads and/or valve positions, it would be best to avoid operating the valve at those positions. Good hydraulic operating practice always needs to be considered.

Mechanical Performance

Laboratory model tests are also used to analyze the required loads upon the various moving parts of a valve, and to determine the valve closure speed, especially under emergency conditions. In addition to considering the normal pressures acting on the valve and components, the design needs to consider the loads from static or moving fluids upon the valves components. It is easy to understand that forces are often quite high, and therefore great care must be exercised in design and selection of components and assemblies.

All disciplines of mechanical design play their role in valves. Valve deflections, stress, sliding and rotating bearing surfaces, and seals dominate the concerns for valves. Valves interrupt the structural continuity of the water conduit (pipe) and therefore the valve body must be capable of transmitting and supporting the pipe loads.

Of particular importance is the possibility of exceeding the strength of valve components, and the consequences of a failure when the valve is subjected to extreme operating conditions, for which it may not have been designed. One possible example is when a valve slams closed due to improper operation or a critical operating component fails. When a valve closes quickly, high loads and stresses are imparted into the valve due to the impact of movable portions of the valve slamming into fixed seats. In addition, the rapid closure results in a transient pressure wave (water hammer) which can result in pressures that are many times the operating pressure to which the valve is normally subjected. While continually subjecting a valve to slamming closures can overstress and fatigue the valve, the ensuing water hammer pressures may be great enough to damage or destroy the valve or the pressure conduit upstream of the valve. The rapid closure of a valve can have consequences that could occur suddenly and often catastrophically.

Solutions:

Valves that have been designed by evaluating the stresses that can develop within the valve under normal operating and extreme operating conditions, and fabricated of suitable strength and corrosion resistant materials, should not result in significant mechanical problems that cannot be addressed by proper and routine maintenance. High head valves of pre-1940 construction with cast one piece bodies may be especially susceptible to cracking of the body if the valve is subjected to very high loads as can be developed when a valve slams shut. Pre-1940 valves were thick bodied and their design did not identify or address stresses and stress concentration. In addition, large diameter, and thick pre-1940 castings were susceptible to sand and slag inclusions and voids within the casting, conditions that can result in stress concentrations and eventual cracking of the body. For valves showing numerous or significant cracking, it is recommended that material testing be performed, or a computer model be developed, to assess the actual and allowable stresses to which the valve can be subjected. If the cracking is significant or the stress levels too high, replacement may be the only safe option.

Checking the rate of closure of a valve under actual operating conditions is often not performed after the initial commissioning tests. Valve closure speeds should be checked on a routine basis to ensure that the valve operator is working properly. However, the tests should not be done without first verifying the transient pressures that would occur during the test, and then assessing whether the valve and pressure conduit can sustain the test water hammer. Modifications have often been made to the equipment or controls of the valve operator that could, unknowingly, result in a valve closing under emergency conditions, with catastrophic consequences.

iii) Seats and Seals

Areas of valves most prone to difficulties are typically the seals and bearings. Valve sealing configurations are too numerous to list, however a typical configuration consists of a seal and seal seating face, both of which need to be in good adjustment and alignment.

Metal-to-metal seals are commonly found in valves of pre-1950 projects, as they were initially considered to be more rugged, and warranted in some circumstances. Modern valves continue to use metal-to-metal seals, especially for high velocity applications, but generally use resilient sealed designs (usually rubber sealed). Debris and cavitation affect the life of both metal or rubber seals, and poor water quality will have a significant impact on the life of metal seals. A valve that sticks, or whose motion is hesitant, may be an indication of a worn or failed seal.

Metal seals wear as a result of increasing frictional loads caused by seal corrosion and the accumulation of sediment between the sliding surfaces. When frictional loads exceed the strength of the attachments, the seal or the base material, the seals fail by tearing, and bind the motion of the valve. Metal seals are straightforward parts that need adjusting, local repairs, or complete refacing or replacing when they are damaged. The difficulty with performing the repair is accessibility to the seals for removal or in-place machining. Depending on the type of valve, its size, and location, it is often accessibility that is the most difficult problem to address, and the valve may require removal in order for repairs to be made. Properly designed corrosion-resistant metal seals should provide 50 or more years of service, if they are not susceptible to damage by cavitation, sediments, debris, or aggressive water quality. Conversely, metal seals that are not corrosion-resistant, or were not properly designed and attached, could have lives of less than 5 years.

Rubber seals, because of their flexibility, provide for a tighter closure than metal seals, and are often considered to be easier to adjust than metal seals to ensure water-tightness. They have a long life expectancy, and in most instances, it is at least 20 years before replacement is even considered, primarily because the seals typically remain wet, and because sunlight does not usually reach the seal, they are not subject to ultra-violet deterioration. Overtightened rubber seals can increase operating torque requirements, and in addition, an over-or under-tightened rubber seal is vulnerable to becoming extruded and pinched off and damaged, or dragged out of its holder. Because of the problems with subjecting rubber seals to high velocities, some concern has been expressed about using rubber seals on high head butterfly valves. As the valve closes, the velocity past the seal increases and may cause the seal to deflect, and possibly be torn free, if the seal attachment is inadequate.

Some modern valves can be equipped with a second seal that is manually extended to engage the sealing surface when the primary seal is to be replaced or repaired. Both modern and older valves may use a movable seal rather than a fixed seal. Movable seals are subject to being frozen in position by rust and sediments, trapped by the seal or within the seal recession chamber. For valves with movable seals, operation and maintenance procedures should include the requirement to periodically move the seals to ensure that they are flushed free of debris, and will operate when needed.

Poorly functioning seals, of any type, increase the need to expend additional operational costs to seal the valve, as well as to minimize its leakage. Seal leakage becomes a concern, particularly when the valve must be closed to allow maintenance or inspection of the equipment or conduit downstream in the dry. Often pressure pots are installed to allow the injection of materials, upstream of the valve, in an attempt reduce the volume of leakage. Such practice, while possibly effective, creates unnecessary risk to operating personnel, as these personnel are trying to quickly inject, under high pressure, a suitable volume of material upstream of the valve.

Solutions:

- For deteriorated metal seals, replacement of the seal with a material less susceptible to corrosion and wear by sediment may be the best solution, as long as the base metal is sound.
- For rubber seals, the method used to attach the seal to the valve body is critical, and varies greatly for each type of valve. Rubber seals will swell slightly after exposure to water for a period of time, and the initial adjustment, if done with the seal dry, will change slightly. Therefore, close attention needs to be given when adjusting a rubber seal.
- For older, rotary-type valves, such as the butterfly and spherical valves, that require significant machining to rebuild and replace their metal seals, it may be possible to use rubber seals instead, but such replacement is dependent on valve size and location (accessibility).

The best solution to prevent seal problem is preparedness, i.e. knowing the condition of the seals for each valve, and understanding the conditions for which the valve was designed. For a site with multiple valves of the same type, seal problems between valves will be varied, and may appear inexplicable or unrelated, although the problem is often attributed to approach flows, and associated velocity and transportation of sediment.

iv) Bearings

Bearings in older valves are typically gun metal, or bronze, and require routine lubrication. Under any use, bearings do wear, and will require replacement over time. Problems occur when their replacement is impeded by the design of the valve, or when lubrication is not provided as needed. Typically, water itself provides some lubricating benefit. However, all good designs will incorporate some means to limit the amount of silt or solids that can reach the bearings. As with any type of bearing, if the bearing is not properly lubricated and free to move, its restricted motion will impede the movement of the valve, possibly resulting in the need to exert more force for the valve to move. The additional forces result in increased stresses, wear, the need for premature repairs or even failure.

Older valves also typically had carbon steel shafts, and the bearings required a lubricating grease, to keep rust from forming, and the mating surfaces from sticking or wearing.

Solution:

- Newer valves take advantage of modern self-lubricating bearing technology. These bearings are a benefit to maintenance personnel, as well as to the environment, as grease injection into the waterway is avoided. If a bearing is to be replaced, a self-lubricating bearing should be used. Some bearing designs even allow the bearings to remain unwatered, allowing for easier inspection, lubrication and repair.

- Newer valves should have both stainless steel shafts and mating surfaces for bearings, or at least stainless sleeves, or overlaid surfaces, so that corrosion is not a problem. For older valves, it may be possible to replace the shafts with similar corrosion resistant materials.

v) Corrosion / Erosion / Water Quality

As with any equipment in and around moist environments, rust or metal deterioration is a primary concern. Often galvanic attack takes place due to contact by dissimilar metallic materials, as may be the condition where rubber or metal seals are attached to the valve body. The life of a valve or certain valve components will also be shortened by mechanical erosion from constant exposure of parts to flowing water containing suspended abrasive materials, or due to cavitation.

Water quality, especially if chemically aggressive, is a problem that attacks nearly all components of a valve, and the worse the water quality, the worse the impact on the valve.

Solution:

- To minimize corrosion of the valve body, avoid maintaining the valve and associated equipment in partially filled or stagnant conditions. Make efforts to maintain their coating systems. Passive (zinc anodes) and active (electrical) programs may be used to reduce the rate of galvanic deterioration. Minerals, or lack of them in the water, can also be sources of corrosion problems, but there is little that can be done to mitigate the mineral content of the water.
- There is little than can be done to minimize erosion of the portions of the valve that are exposed to abrasive materials in the water. For new valves, the use of abrasive-resistant materials is an option. For existing valves it may be possible to coat the affected surfaces with a protective material. A valve manufacturer should be consulted regarding the use of abrasion resistant coating systems. A simple means of minimizing the problems caused by erosion damage is to routinely monitor and document conditions and rates of change, thus allowing the problem to be addressed before it results in a failure of any of the valve's components.
- For erosion caused by cavitation, depending on the composition of the valve component, it may be possible to replace the lost material by adding weld metal. The best way to minimize cavitation is by not routinely operating the valve in a position that promotes cavitation. Often a slight change in the valve opening may be all that it takes.
- There is little that can be done to control water quality. The best solution to address the problem is to construct a valve using materials that are not affected by the chemical composition of the water at a specific site.

vi) Operators

Valve operators deserve important considerations because it is the operator that controls the valve. Operators, whether manual, electric, hydraulic, or pneumatic, work as an integral part of the valve. The condition of the operator, and the keys and pins that transmit torque on shafts, are no less important than the valve's own shafts, drives, or stems. A failure of the operator, or the connecting pins between the operator and the valve, could result in an inability to control the valve and stop the uncontrolled release of flow, or could result in the valve slamming shut, with possible catastrophic consequences.

Operator failures, or valve failures caused by the operator, are the result of excessive forces being applied to overcome the valve's resistance to movement. Valve operators should be equipped with shear keys sized to prevent the transfer of excessive forces. Often the cause of the excessive force is not the operator or operating condition, but some condition occurring within the valve (e.g. rusting, sediment depositions, damaged seals).

Assuming that the valve has been suitably sized for the loads intended, and assuming that the valve has been properly maintained, the operator system as a whole should theoretically have an indefinite life, while some components and subsystems may be replaced numerous times due to routine wear or system upgrades. Unless a valve operator has been operated beyond its rated capacity, the upgrade or replacement of a operator is often done as a convenience, particularly if the upgrade or replacement provides better operational control of the valve or allows untended operation of the valve.

If a valve is considered to be a critical feature, i.e. required to prevent the uncontrolled release of an impoundment, then the valve operator should be connected to an alternate (backup) power source to ensure that the valve can be operated as needed and when needed even with the loss of the primary power supply.

Solution:

- To minimize the transfer of potentially damaging forces, do not randomly increase the size or strength of shear keys. If a valve and operator have been properly designed and sized, and if any key within the valve or operator system fails frequently, the cause of the increase in operating forces needs to be determined. Installing larger or stronger shear keys should not be the solution to the problem.
- Gearboxes and all bearings within the operator and system transferring torque to the valve should be routinely inspected and lubricated. Lubrication oils should be tested for presence of water and metal filings, either of which are indicators of a potentially serious problem.
- Verify the valve closure speed to ensure that an emergency closure of the valve will not result in damage or failure of the valve or pressure conduit. Checking the

rate of closure of a valve under actual operating conditions is often not performed after the initial commissioning tests. Valve closure speeds should be checked on a routine basis to ensure that the valve operator is operating properly, but the tests should not be done without first verifying the transient pressures that would occur during the test and then assessing whether the valve and pressure conduit can sustain the test water hammer. Modifications have often been unknowingly made to the equipment or controls of the valve operator that could result in a valve slamming closed under emergency conditions, with catastrophic consequences.

vii) Valves with Self-Closing Tendencies

Usually valves that rotate 90 degree to control flow (spherical, butterfly) have self-closing tendencies, due to flowing water trapping air in chambers, although some needle and horizontally sliding cylindrical type valves (needle, sleeve) may also exhibit self-closing tendencies. Unanticipated and uncontrolled movement of a valve could have catastrophic impacts on the valve, the pressure conduit, plant operation, and personnel.

Valves that operate by the balancing internal hydrostatic pressures as a means to move the valve (some types of needle valves) have failed catastrophically as a result of sudden uncontrolled movement (either opening or closing) as a result of improper operation and maintenance of the valve or control system.

Solutions:

Knowledge and understanding of how individual valves work is the best measure to prevent a valve from slamming closed, self-closing or opening. Ensuring that all is done to recognize this possibility, and that everything is done to prevent it from happening, cannot be overemphasized. Plant personnel should recognize if any valve within their plant has the tendency for self-closure, so that it is not overlooked and the appropriate operation and maintenance procedures can be undertaken to prevent it from happening. Where valve designs utilize the balancing of internal hydrostatic pressures as a means to move the valve, close inspection and maintenance is a must to ensure that all of the pressure balancing valves and piping function properly, do not leak excessively, and are not obstructed with sediment or other debris. In addition, such valves require a sound understanding of operational procedures and the forces that are being developed, internal to the valve, when the procedures are followed, or not followed.

viii) Large Valves and Deflection

As valve size increases, the designs become more prone to larger deflections. In small valves, the bodies and components are disproportionately and, typically, extra thick, because they need sufficient material thickness for machining, manufacturing, and installation. As a result, the stresses within the valve body are quite often very low and the valve is also very stiff.

As valve sizes increases, maintaining the same (material thickness) proportions as that for a small valve would not be cost effective, or practical. Larger valves have components sized with thickness according to appropriate stress values. While stresses may remain low, deflections can become the controlling design criteria. Properly engineered, larger valves will consider deflections, and incorporate stiffeners, or flanges, as needed to reduce deflections.

Deflections can cause problems in large valves. There are a number of examples of this: for example when the sun heats only one side of a valve and penstock and causes uneven expansion; or a large butterfly disk deflects when closed under pressure, but is nonetheless safely stressed; or the shape of a butterfly body under full pressure is very round, whereas the valve becomes egg shaped when dewatered. Problems associated with deflections include: poor seal seating resulting in leakage; interferences between moving components; unsymmetrical load and uneven wear of components and the possibility of the valve being jammed in either the open or closed position.

Solutions:

There is little that can be done to correct excessive valve deflection that results in increased leakage or other problems with the valve. Deflection is a result of design and material thickness. If problems are significant, replacement is possibly the only option.

b) Valve Specific Problems

The following section describes problems common to a particular type of valve, including:

i. Butterfly Valve.
ii. Fixed-Cone Valve.
iii. Hollow-Jet Valve.
iv. Needle Valve.
v. Internal (Pressure) Differential Valve.
vi. Sleeve Valve.
vii. Spherical Valve.
viii. Poppet Type Pressure Relief Valve.
ix. Valve Operators.

i) Butterfly Valve

Primary problems are typically the seals and bearings. Some valves can be taken apart while others are not so easy. Some designs allow bearings to be replaced *in situ*, but typical low cost AWWA style valves do not. Most resilient seals can be replaced *in situ*. For valves fabricated before 1970, the metal seals and seats require either the removal of the valve, or for large diameter valves, removing the valve from service and *in-situ* field machining to rebuild or repair the seats. New valves can be designed with metal seals and seats that can be replaced in situ. In large valves, adjusting the seals to be watertight is difficult, as valve deflections under water pressure will differ from those present when the seal is adjusted in the dry.

Older style valves often had centerline, metal-to-metal seal designs interrupted by shafts, and seals and seats that are difficult to maintain. There have been successful conversions where the older metal-to-metal sealed valves were converted to rubber-sealed valves.

The most dangerous aspect of butterfly valves is their tendency to slam closed should there be a failure of the valve operator or the linkage shaft, pin or key connecting the operator to the valve. The hydraulics of water flowing over the disk of a butterfly valve tend to force the disk closed when the valve is less than one-third open. A properly designed and functioning valve operator forces the disk to close from the full to one-third open positions, then for the last third of the opening, the operator holds back the disk to prevent it from slamming closed.

In some facilities butterfly valves are exposed to high water velocities, resulting in excessive head losses (energy loss) and induced vibration. Flow velocities through butterfly valves should not exceed 15 to 20 feet per second.

ii) Fixed-Cone Valve

The design of the hollow-cone, hollow-jet, and sleeve valves evolved in the 1930s-1960s from the interior differential needle valve. As opposed to the needle valve, the hollow-cone, hollow-jet, and sleeve valves do not operate under the principle of balancing the forces within the valve. They all require an external power source to move and hold the valve in position.

Early fixed-cone valves were prone to vane cracking and failures. Employing design criteria, established by Albert Mercer to judge if vanes are vulnerable to failure, all but eliminated this problem. The fixed-cone valve operates in the same manner as a sleeve valve. For common problems, see common problems associated with sleeve valves.

Unique to this type of valve is a squeal that can be heard at, or near, but not at the full closed position. While a normal occurrence for this type of valve, it is not recommended to operate the valve for prolonged periods in a position where the noise occurs.

If cavitation damage occurs, check design conditions against operating conditions, and modify operating parameters to avoid operating positions that result in cavitation damage.

iii) Hollow-Jet Valve

The hollow-jet valve operates in the same manner as a sleeve valve. For problems common to the hollow-jet valve, see common problems associated with sleeve valves.

iv) Needle Valve

The needle valve was developed in the mid to late 1800s by Doble as a device to control high head flows to Pelton turbines. The valve's needle (or plunger) was unseated and withdrawn to increase flow, with the needle being operated by a mechanical external operator. In 1905 the Bureau of Reclamation utilized a redesign of the needle valve as a means of controlling the free discharge from high head dams. The redesigned valve operated based on pressure differentials that occurred inside the valve, as a result of flowing water, and is generically identified as an internal differential valve.

Needle valves are also used to control the free discharge jet on the runovers of Pelton turbines.

v) Internal (Pressure) Differential Valve

The internal differential valve was designed to operate using reservoir pressure without the use of external operators or power sources. To produce movement of the valve's plunger, the pressures within the valve are unbalanced. The valve is a positive seating closure device with closure being accomplished by a large diameter plunger being forced against a smaller diameter opening.

The early internal differential valve, known as the Ensign valve, was located on the upstream side of the dam. The Ensign valve was short-lived and led to the development of the balanced needle valve, which was located at the end of a pressurized conduit, and then to the internal differential needle valve in 1927.

The Johnson valve (later the Lanier-Johnson valve, early 1920s) was developed as a refinement of the Ensign valve, and was intended for in-line applications for the emerging high head hydroelectric industry in the United States. Beyond the intended use for the valves, the main difference between the Ensign and Johnson valves was how the pressure differences within the valve were communicated.

The Johnson valve, the balanced needle valve, and the internal differential needle valve were all hydraulically operated, based on a simple principle of utilizing the pressure differentials between the upstream and downstream sides of the valve. The pressure differentials were developed as a result of velocity changes, with the highest velocities, resulting in lower head pressure, located on the downstream side of the valve. The internal pressure was controlled using a pilot valve in the top of the valve body, or through a pilot valve located in the nose of the plunger.

The use of the balanced needle valve and the internal differential needle valve was generally associated with Bureau of Reclamation dams in the western United States through the 1940s, although most of the balanced needle valves have since been replaced by hollow cone or sleeve valves. Most of the Lanier-Johnson valves installed at hydroelectric projects in the United States pre-1940s have also been replaced with other types of control or isolation valves. As of 2002, the Lanier-Johnson valve was still being manufactured in England and used in water supply dams in developing countries where sites are remote and power may not be available.

Although the internal differential needle valves and Lanier-Johnson valves are no longer in prominent use in the United States, there are still some in service. The following list of problems include some of the reasons that the valves were replaced or removed from service:

- Accumulation of sediment in the valves used to control the position and operation of the plunger.
- Deterioration of metal seals and sliding surfaces due the (pre-1940s) lack of high quality corrosion and erosion resistant materials.
- Erratic movement of the plunger due to deteriorated seals.
- Increased leakage into the internal pressure chambers, due to seal deterioration, resulting in the inability to maintain the position of the plunger or uncontrolled movement of the plunger.
- Field modifications made by the installation of valves in the control piping system in an effort to control seal leakage and movement of the plunger.
- Installation of electrically operated valves within the control piping system in a attempt to automate the operation of the valve in conjunction with hydroelectric turbine control. This can exacerbate the problem with the control of the movement of the valve plunger.
- Complexity of operation when rewatering the valve. Improper venting when rewatering could result in air bound chambers within the valve which upon compression could cause sudden uncontrolled movement of the plunger.
- Uncontrolled movement of the valve plunger which could result in the valve slamming closed, possibly causing the catastrophic failure of the valve or connecting penstock.

In 1940 the original design for the Lanier-Johnson was changed to replace the hydraulically operated pilot valve with an external mechanical operator for the pilot valve. The plunger was mechanically locked to the external operator to prevent a sudden closure of the plunger. The development of better corrosion and erosion resistant materials after 1960 was reported to have eliminated the valve's problems with seal deterioration and poor control of the position of the plunge.

vi) Sleeve Valve

Like the hollow-cone, and hollow-jet valves, sleeve valves evolved from the interior differential needle valve and required an external power source.

In the sliding sleeve type valve, the sleeve has both upstream and downstream seats and seals. These seals and seats can cause problems, but solutions are as unique as the designs. Contact a manufacturer who produces valves with the seal design you have, for possible solutions. To minimize problems, adjust or replace seals and packing, and check for loose retaining rings and seat rings. Uneven or erratic sliding of the sleeve can result from: misaligned operators or hydraulic cylinders; uneven pressure or force; packing being unevenly tightened or too tight; or debris lodged between sleeve and body clearances. If the valve will not open check that packing is not too tight.

For multi-jet valves, if the valve does not pass as much water as in the past, check for plugged multi-jet holes.

vii) Spherical Valve

Common problems for spherical valves are similar to those associated with butterfly valves. The sliding (movable) metal seats of the spherical valves are prone to jamming or sticking. Routine exercising seems to be the best remedy against problems. Metal to metal seats are often used on high pressure valves, which is typically what this class of valve serves, and are prone to maintenance problems from nicks and scratches.

Like the butterfly valve, the most dangerous aspect of a spherical valve is their tendency to slam closed should the valve operator or any shaft, pin or key fail.

viii) Poppet Type Pressure Relief Valve

This valve is an older style turbine pressure relief or bypass valve. Today it is, typically, not a valve of choice, and is usually substituted with the fixed-cone or multi-jet sleeve valve.

Poppet valves usually wear out over time. Typically, energy dissipated by this valve wears out valve parts or their immediate downstream waterway by cavitation erosion or the direct impact of the discharging jet. This requires the replacement of bearings and seals, and frequent inspection of the valve body, stem, covers, and plug for evidence of cracks or wear.

ix) Valve Operators

Valve operators should be maintained in the same manner as the valve. Under normal conditions, the required force, torque, oil pressure and motor amps should be checked as appropriate in order to trend the health of the valve and operator over time. This will be useful information when trouble shooting.

Weather and temperature can affect operators, due to effects on oils and grease. Ice can form on the operator or valve itself. Manual operators are vulnerable to worn damaged, parts, loose or broken fasteners, keys and pins.

Electric actuators are vulnerable to the same problems as manual operators along with misadjusted limit or proximity switches, motor problems, relay and control problems, and power supply problems.

Hydraulic operators are vulnerable to all of the previously mentioned problems along with leaky seals, contaminations blocking oil flow, and environmental oil spills. It is recommended that good records be kept, pressures be monitored and trended, and that as many test ports as practical be provided where pressures can be monitored during troubleshooting.

6.3.3 Corrective Measures

For this section of the guidelines, the solutions and corrective measures are described in conjunction with Section 6.3.2 – Problems.

6.3.4 Opportunities

Opportunities for improving the performance of a valve are somewhat limited to improving the valve's discharge characteristics, or improving the control of the valve's operation. Improving the means of controlling a valve may not require the removal of the valve and may not be cost prohibitive to implement. Replacing a valve only to improve its discharge characteristics is not normally done for economic reasons, unless the valve needs to be replaced due to its condition, or the hydroelectric project is undergoing a significant upgrade and outage. The cost to replace a valve will not only includes the cost of the valve and installation, but the cost of lost generation if the valve is located upstream of a hydroelectric turbine, or the costs to dewater if the valve is used to control discharges from a dam.

6.3.5 Case Histories

No. 1 Safety & Operational Concerns
Carolina Power & Light (Kleinschmidt Associates, 1997)

The Walters Hydroelectric Project is located in Waterville, NC, and was constructed in 1927. The powerhouse contains three Francis turbines and each turbine had a Lanier-Johnson (needle) valve and butterfly valve used to control and isolate flow to the turbine. The turbines and valves have a discharge capacity of 750 cfs under a static head of 853 ft and with a transient surge head of 1,265 ft. The powerhouse was located approximately six miles downstream of the dam.

The Lanier-Johnson valves had seals and sealing surfaces that had deteriorated beyond repair and extensive efforts were required to dewater the penstock and turbine. In addition, there were increasing problems with the ability to control the motion of the valves, and one of the valves had slammed shut on at least two known occasions, resulting in a crack that extended one-quarter of the way around the valve body. The Lanier-Johnson valve was used as the primary means to isolate flow to the turbine. The turbine wicket gates were susceptible to erosion damage when the wicket gates were closed and the turbine was continually subjected to full static head. The butterfly valve was only used as a maintenance valve, because the operator used to close the valve under emergency closure conditions, had been modified and rendered inoperable, due to its condition. In addition, the seals in the butterfly valves were deteriorated and unable to allow the dewatering of the penstock, without extensive efforts to seal the valve.

Investigations and materials testing were performed and indicated that the Lanier-Johnson valve bodies might not be able to sustain the stresses associated with the transient surge pressures, due to cracks and sand inclusions in the casting. Based on the tests, it was determined that the Lanier-Johnson valves should be replaced for safety purposes. Because the replacement valve needed to function only as an shutoff device, the evaluation included replacing the valve in-kind or with a spherical valve, with the replacement valve located in the same place as the existing Lanier-Johnson valve. Butterfly valves were considered unsuitable for use at the site because of the head to which they would be subjected, the velocity through the valve under emergency closure conditions, and the increase in headloss through the valve. Spherical valves were installed when three existing suitably sized valves were found in storage at a hydroelectric project whose construction had been terminated.

The new spherical valve had slightly reduced headlosses, resulting in an increase in generation, because it had a better discharge coefficient than that of the Lanier-Johnson valve. The spherical valve was equipped with maintenance seals in addition to the primary seals, allowing the primary seals to be replaced without the need to dewater the penstock between the spherical and butterfly valves. The maintenance seals eliminated the need to replace the seals on the butterfly valve

6.3.6 Collective Knowledge

a) General Information and Design

1. American Society of Mechanical Engineers (ASME). Hydropower Technical Committee. (1996). *The Guide to Hydropower Mechanical Design: Chapter 7.* HCI Publications, Kansas City, MO.

2. Creager, W.P. & Justin, J.P. (1950). 2nd Edition. *Hydro-Electric Handbook.* John Wiley and Sons, New York, NY.

3. Lewin, J. (2001). *Hydraulic Gates and Valves.* Thomas Telford, London.

4. Zipparo, V. & Hasen, H. (eds.). (1993). 4th edition. *Davis' Handbook of Applied Hydraulics.* McGraw-Hill, New York, NY.

b) Design Standards

Prior to 1930, there were few standards in general use for the design of valves, as most gate design standards were developed by the design firm or the engineer of record. Federal agencies such as the Army Corps of Engineers and the Bureau of Reclamation have developed standards of their own. Commonly used design standards include:

1. American National Standards Institute (ANSI) (various standards). http://www.ansi.org
2. American Society of Mechanical Engineers (ASME). (2004). *2004 Boiler and Pressure Vessel Code: Section VIII.* ASME, New York, NY.
3. American Waterworks Association (AWWA). Denver CO. (various standards) http://awwa.org
4. German Institute for Standardization (DIN): German Institute for Standardization (various standards)

6.3.7 Technical References

Kleinschmidt Associates. (1997). Personal Communications, Kleinschmidt Associates, Columbia, SC.

6.4 Flashboards

6.4.1 Function

Flashboards are defined as a mechanism to raise the impoundment behind a dam by the installation of a barrier on top of the spillway. Originally (prior to the 1930's), flashboards were used to temporarily raise the level of an impoundment, often during summer months, while providing the dam with the designed spillway discharge capacity during high flow periods (i.e. spring time) when the flashboards would not typically be installed. As electric power demand increased, owners and operators would leave flashboards on spillways for longer periods of the year in order to provide additional head for the generation of electricity. In some instances, the flashboards would be left in place throughout the year, where they provided the benefits of increased head and generation, but were designed to fail at a specified water level or flow, allowing the original discharge capacity of the spillway to be realized during high flows.

Flashboards can vary in height from a few inches to ten to twelve feet. Simple flashboard systems typically consist of vertical steel pins or pipes inserted in sockets in the spillway every one to seven feet with horizontal timbers or planks spanning the pins that protrude from the sockets. Pins or pipes may range from one to six inches in diameter and are typically designed to fail in bending when the impoundment elevation reaches a prescribed level, allowing increased spillway discharge capability.

For flashboards with heights in excess of six feet, the flashboards may be inclined or vertical, with diagonal wood struts used to support the flashboards. Inclined flashboards and flashboards that are supported with struts are intended to fail by being manually released or tripped. Less typical flashboard arrangements also exist (Creager and Justin, 1950; Barrows, 1927).

Stanchion stoplogs, vertical steel beams with wood planking spanning horizontally between the beams, are often considered to be a type of flashboards. The upper part of the stanchion engages a manually operated releases mechanism, and the bottom end is either hinged or sets in a pocket or against a vertical lip. Stanchion stoplogs can reach heights in excess of 20 feet, but because of the need to provide a top support for the stanchion beams, they are installed in bays with widths that are generally less than 50 feet. Stanchion stoplogs are an inexpensive way to construct a dam with a high discharge capacity, but because the stoplogs are either in-place or released, they are not a good means for controlling normal flow releases from a dam; therefore they are often augmented by mechanically operated gates that are used to discharge normal flows.

Having a properly designed flashboard system is important for two reasons. The first reason is that flashboards allow the impoundment to be operated at a higher elevation. If there is hydroelectric generation at the facility, the higher impoundment elevation relates to a higher gross head and increased generation revenues for the facility. If the impoundment has recreational use, the higher pond elevation provided by flashboards provides better overall recreational uses including enhanced boating, fishing, access to docks and public boat ramps. The elevation of the top of the flashboards should be within the limits of the project license or permit. Where hydroelectric generation exists, it is in the owner's best financial interests to maximize the period of time the flashboards are in place with the impoundment maintained at the higher elevation in order to maximize the generation. Premature failure of the flashboards will result in a reduction of generation revenue, increased labor and expenses in order to re-install the system and possible loss of recreational benefits. Depending on the height of the flashboards, frequent and premature loss of boards and resultant lowering of the impoundment could also have environmental consequences by impacting adjacent wetlands and shorelines during critical times of the year when fish and wildlife habitat is flourishing. Also, frequent loss of flashboards can result in an unacceptable amount of debris (i.e. flashboard timbers) being introduced into the river and floating downstream.

The second reason for having a properly designed flashboard system is to ensure the timely failure of the boards under high flows, thereby providing increased flood discharge capability over the spillway, and protecting the stability of the water retaining structures. If the flashboards do not fail at their design elevation, this could result in: water retaining structures being exposed to water pressures that may be in excess of their design limits; higher than expected impoundment elevations during high flows with the potential for over topping other water retaining structures such as embankment dams resulting in their failure; and higher than expected impoundment elevations that could cause incremental flooding or erosion upstream.

There are some documented examples of flashboard systems that did not operate as designed. The following is an excerpt from the Guidelines for Design of Dams published by the New York State Department of Environmental Conservation in 1989:

> "In 1939 flashboards were placed across the spillway of the 40 foot high Tilson Lake Dam in such a manner as to greatly reduce the spillway opening. Storm flow caused dam overtopping which eroded the earth slope in front of the 100 foot wide, 30 foot high concrete core wall. Failure of the core wall resulted in a tremendous amount of erosion to farm land, loss of farm machinery, chickens, several local bridges and basement flooding. The dam was rebuilt and failed in 1955 because flashboards were again in place and did not fail during storm flow."

6.4.2 Problems

The following are typical problems that are encountered with simple flashboard systems:

a) Flashboard arrangement was not designed or the design is not documented.
b) Flashboards do not operate as designed.
c) Reduced net income.

a) Flashboard arrangement was not designed or the design is not documented

Many owners/operators adopt a certain flashboard arrangement because it has been used historically. One of the reasons may be that the design has never failed and, therefore, the assumption is that it must be a "good" design. However, flashboards that never fail are not necessarily a "good" design. Documentation should exist to validate the current flashboard design and the level at which the flashboards were designed to fail so as to avoid upstream flooding.

b) Flashboards do not operate as designed

Assuming that a design was performed for the flashboard system, there are other variables that can influence actual failure points, compared to theoretical failure points. They include: excessive ice thrust; the ultimate strength of the pins being different in design; excessive backwater accumulation against the downstream face of the boards; materials and configuration used in the field that are different from design assumptions; deterioration of materials; ice jacking (i.e. vertical lifting of the boards); debris impact; and leakage/freezing during winter.

c) Reduced net income

Flashboard systems that are designed to fail require re-installation at a later time. Failure, or tripping of the flashboard systems, especially if there are a frequent or excessive number of failures, will result in reduced revenues from generation, and increased operation and maintenance costs due to the need to re-install the flashboards. There are two economic factors that should be considered by owners: the annual expense to maintain the flashboard system, and the lost generation that occurs when the boards do fail. If the frequency of board failure on an annual basis becomes excessive, then increased expenditures are required by the owner to replace the boards. In addition, when the boards do fail, either high river flows or impact by floating debris are usually the cause. If the boards fail during high flows and unless the facility is equipped with extra flood discharge capacity allowing the impoundment to be promptly lowered temporarily below the crest, the owner will usually have to wait until high flows subside before the boards can be replaced safely. In many instances, this can result in several months of operation at a reduced head, resulting in significant lost generation revenue.

6.4.3 Corrective Measures

The following are corrective measures associated with the indicated problems.

a) The flashboard arrangement was not designed or the design is not documented

Owners should have documentation to validate the design of the flashboard arrangement. Depending on the flooding impacts upstream and downstream, the risks associated with an improperly designed system can be great. The analysis and design of flashboards is not difficult or complex. If the design is not documented then an analysis should be performed.

b) Flashboards do not operate as designed

The causes for a flashboard system not operating as designed are many, and some may be dependent on the site geometry and river conditions. As discussed in this section, the following are some of the more common causes of flashboards not operating as designed:

i. Excessive Ice Thrust.
ii. Inappropriate Strength of Materials.
iii. Excessive Accumulation of Water Against the Downstream Face of the Boards.
iv. Change of Materials or Configuration.
v. Deterioration of Materials.
vi. Ice Jacking.
vii. Debris Impact.

viii. Leaking and Freezing During Winter.

i) Excessive Ice Thrust

For dams in colder climates, the formation of an ice sheet in the impoundment is a normal winter occurrence. When the ice sheet expands due to thermal changes, the pressure exerted upon the simple flashboard arrangement will usually cause them to fail. One solution is to install an underwater circulator on the upstream side of the flashboards. This resembles a small outboard electric trolling motor with a submerged propeller that circulates the water immediately upstream of the flashboards, thus preventing the formation of ice, and leaving a buffer zone between the ice sheet and flashboards. The ice sheet can expand without applying any force to the boards. Another solution is to install an air bubbler system along the upstream side of the spillway for the full length of the boards. This provides the same benefits as the underwater circulator and is best suited for longer dams where a circulator may have limited impact. Other solutions include manually chipping or cutting of the ice to free it from the flashboards.

ii) Inappropriate Strength of Materials

Due to the need for replacement of materials in a failed or tripped flashboard systems, especially if the materials need to be replaced frequently, there is often a tendency to use heavier or stronger materials to minimize the number of failure or tip events. The simplest solution is to determine what material strength and sizes were specified for the original design, and to replace the flashboards as they were designed.

Steel strengths used in the design of pins and pipes are generally taken from published sources such as American Society of Testing and Materials (ASTM). These sources typically use minimum ultimate strengths, while the actual ultimate strength will be larger, causing the failure point of the pins to be higher than expected. Some foreign mills and suppliers provide materials that do not conform to ASTM Standards. When purchasing pins and pipes for use in the construction of flashboards, mill certifications should be required to verify the strength of the materials being supplied. One way to determine the strength of the steel pipe is to take a series of test coupons from the steel pins (either from the pins that exist on the dam or from pins that will be installed at a later date), test for the actual ultimate strength, and incorporate these values in the design. Another solution is to machine the pins to a specific diameter that corresponds to failure at a predetermined pond level. This will usually require machining various diameters, followed by actual field tests to determine the correct machined diameter that corresponds to the desired pond elevation where failure is required.

iii) Excessive Accumulation of Water Against the Downstream Face of the Boards

During the design of the flashboard arrangement, it is usually assumed that no water exists on the downstream side of the flashboards to resist the tendency of the pins to fail. However, in special circumstances, the spillway surface immediately below the boards is relatively flat and does not drop off. As a result, water will tend to accumulate in this area when water is spilling over the boards, preventing the pins from failing as designed. One option is to modify the configuration of the spillway to prevent water from accumulating, but this will usually be costly. Other options are to modify the design to use a smaller pin size to account for the restraining effects of the water below the boards, or to move the flashboards closer to the downstream face of the spillway.

iv) Change of Materials or Configuration

Owner/operators may not always appreciate, or understand, the importance of the flashboard operation. The thinking is that failure of the boards is an undesirable event and must be prevented. Case History No. 1, later in this section, describes what can happen when flashboards are strengthened beyond their original design when they should not have been. In addition, what may have started out as a designed system several years before, can result in a different system than was originally designed. For example, the original design requirement to use 2 inch diameter standard strength pipe was inadvertently modified when 2 inch diameter heavy walled, stronger pins were more readily available. This resulted in a flashboard system failing at a much higher impoundment elevation as compared to the original design. Another problem that can occur is when steel pins fail as designed and the owner/operator, in an effort to reduce costs, salvages the failed pins, straightens them and then re-installs them. When the pins are straightened, they undergo a metallurgical process known as stress-hardening in which their strength becomes greater than it was originally. This will result in the pins failing at a higher pond elevation than was originally designed for or intended. Failed pins that have been straightened should not be reused for this reason.

v) Deterioration of Materials

Where flashboards are designed to remain in place all year, deterioration of existing members can become a problem that needs to be addressed. Typically, the symptoms of deterioration are premature failing of the flashboards or excessive leakage. Steel corrodes and wood rots. As the corrosion or rotting progresses, eventually something will fail. All structural elements should be inspected on an annual basis with deteriorated members replaced as required.

vi) Ice Jacking

During winter operations where the impoundment ice sheet attaches to the upstream side of the flashboards, ice jacking can occur. As the impoundment elevation raises, such as during increased river flows, the ice sheet will also raise and lift the flashboards with it, either causing premature failure of the boards, or allowing ice build-up, which could prevent the flashboards from failing as designed. This can be prevented by using the underwater circulator or air bubbler as discussed in i) above.

vii) Debris Impact

Debris impact is a common cause of premature failure of flashboards. This is common, especially during springtime, when elevated river flows collect debris, such as large logs, from along the shore, and carry them downstream to the dam, where they impact the flashboards. Ice sheets that form in the impoundment can also breakup and impact the flashboards. During the summer months debris can impact the boards, causing them to fail. Every river will exhibit it's own problems in this regard. Some rivers are not as likely to contain much large floating debris and/or ice, while other rivers can contain a large amount. Regardless of what impacts on the flashboards, large debris and/or ice can cause either isolated or complete failure to the flashboards. Floating log booms might help divert debris to one end of a spillway, where it can be collected or passed over the spillway, or large floodgates can be operated to divert some debris away from the flashboards. However, there is no universal solution that can be implemented to completely avoid this problem in a cost effective manner.

viii) Leaking and Freezing During Winter

Leaking and freezing of flashboards during the winter months can create a serious problem for dam owners and operators. Typical flashboard arrangements, consisting of steel pins and planking, are very prone to leakage. While minor leakage during spring, summer or fall does not generally present operational problems, this can present a significant problem in the northern climates during subfreezing temperatures. If the flashboards are not completely sealed of all leakage, the leakage will freeze, encapsulating the flashboards with ice and preventing their ability to fail as designed. Solutions include: removal of the flashboards during the winter months; periodically clearing ice from the flashboards, using steam or other deicing equipment; or taking additional measures to seal any leakage, such as installing plastic sheets on the upstream face of the boards.

c) Reduced net income

When the expenses and lost revenues become a problem with an existing simple flashboard system, there are a number of options available to owners to address these areas. While the following options are not an exhaustive list, they do represent some of the more typical alternatives owners have considered and implemented in recent years. They include:

- Manually Trippable Wood Flashboard System.
- Shear Pin Trippable Flashboard System.

- Hydraulically Operated Flashboard System (Crest Gates).
- Pneumatically Operated Flashboard System (Rubber Dam).

Appendix C, Table C-4, Flashboards Systems compares the advantages and disadvantages of each of the systems identified above. The subsections that follow describe the operation of the individual systems:

i. Manually Trippable Wood Flashboard System.
ii. Shear Pin Trippable Flashboard System.
iii. Hydraulically Operated Flashboard System.
iv. Pneumatically Operated Flashboard System.

i) Manually Trippable Wood Flashboard System

This system is very similar to the simple system. It consists of vertical steel pins with horizontal wood boards spanning the pins, and also includes systems identified as stanchion stoplogs. The pins and boards are designed not to fail, but instead to trip. Tripping is accomplished by manually pulling the first steel pin, usually located at one end of the spillway. Once the first pin is pulled, the overlapping nature of the boards allows the sequential rotation of the boards across the spillway, opening up most of the spillway to discharge the high flows.

Figures 6.4-1 and 6.4-2 show a somewhat more sophisticated system that was designed to be tripped from both ends of the spillway, with two separate vertical levels that can be tripped independently, depending on level of flows. Resetting the boards is accomplished by lowering the impoundment below the spillway crest and manually resetting the boards. The type of flashboard arrangement as shown in Figures 6.4-1 and 6.4-2 is generally suitable for vertical heights of less than 5 feet.

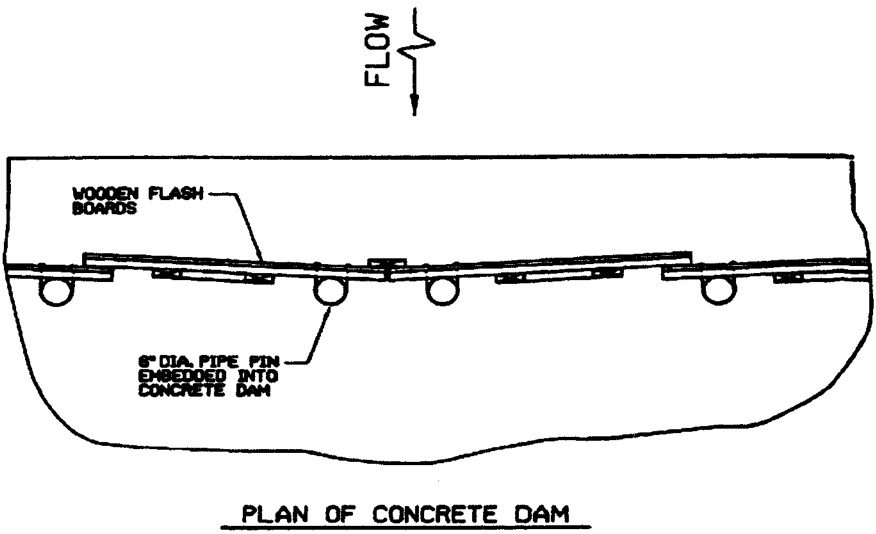


Figure 6.4-1 Plan of Spillway Showing the Manual Trippable Board System (courtesy of Brookfield Power New York)

Figure 6.4-2 Photo of a Two Level Manually Trippable Flashboard System with the First (Lower) Level Tripped (courtesy of Brookfield Power New York)

Variations of this arrangement can be used for taller flashboards. These include installing vertical or inclined panels (wood or steel), with one or more diagonal wood struts to support the boards. Operation (or tripping) of this arrangement is accomplished by pulling a cable which removes the diagonal strut.

ii) Shear Pin Trippable Flashboard System

This system consists of vertical steel pins hinged to the spillway crest, with steel or wood panels attached to and spanning the vertical pins. Tripping is accomplished by designing a shear pin, located in the hinge assembly, to fail at a stress associated with the water surface elevation where tripping is desired. Once the shear pin fails, the flashboards fold downstream and typically lay flat on the spillway surface, allowing the full spillway surface to be available to discharge high flows. Resetting the boards is accomplished by lowering the impoundment below the spillway crest, manually raising the hinge board assemblies, and replacing the shear pins. The system shown in Figures 6.4-3 and 6.4-4 is one variation of the shear pin trippable system and is used as an example. The reader is referred to other sources for variations of this system (Creager and Justin, 1950; Barrows, 1927).

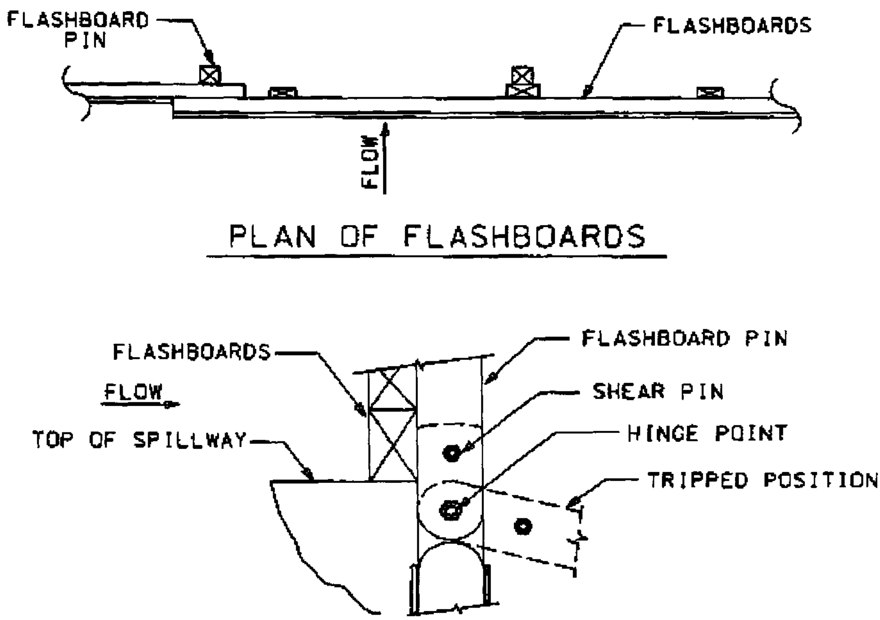


Figure 6.4-3 Plan and Section Showing Shear Pin Trippable Flashboard System (courtesy of Brookfield Power New York)

Figure 6.4-4 Photo Showing the Shear Pin Trippable Flashboard System (courtesy of Brookfield Power New York)

Variation of this design includes steel or wood panels that are hinged along the base. This arrangement requires a cable fastened between the top of each pin and the upstream face of the spillway. The cable contains a shear link that is designed to shear at a predetermined load, corresponding to the desired impoundment elevation where tripping is required.

iii) Hydraulically Operated Flashboard System (Crest Gates)

This system consists of steel flashboard panels hinged at the spillway crest. Typically, these panels are less than 5 feet high with lengths upwards of 100 feet. Hydraulically actuated rams are spaced intermittently across the spillway, fastened between the downstream face of the panels and spillway surface downstream of the hinge point (Figure 6.4-5). Raising and lowering the flashboard system is accomplished by use of the hydraulic rams on the downstream side of the panels. In lieu of the downstream rams, some designs use a torque tube along the base of the panels that is operated by a hydraulic ram at one end of each panel. The hydraulic flashboard has evolved into large crest gates with heights up to 25 feet and panel lengths upwards of 80 feet.

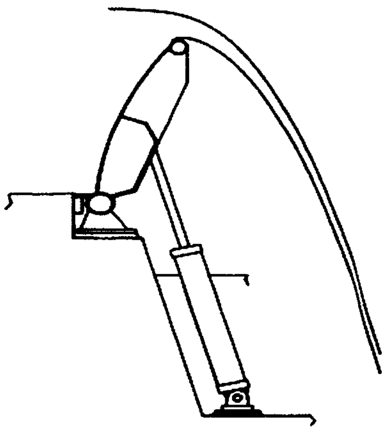


Figure 6.4-5 Hydraulically Operated Flashboards (courtesy of Brookfield Power New York)

iv) Pneumatically Operated Flashboard System (Rubber Dam)

This system is comprised of individual rubber air bladders controlled by a blower or compressor. The bags or bladders are fastened to the spillway crest using a steel clamping system. Some pneumatic flashboard manufacturers utilize only the bladder as the flashboards, while others add a steel plate on the upstream side of the bladder hinged to the spillway crest (Figure 6.4-6). The pneumatic flashboard system is maintained by pressurizing the rubber bags or bladders. Lowering or raising the flashboards is accomplished by automatically or manually deflating/inflating the bladders.

6.4.4 Case Histories

No. 1 Flashboard Operation Misunderstanding Gore Mountain Dam (NYSDEC, 1989)

Many field maintenance personnel do not understand the need for flashboards to fail when the depth of flow over the flashboards reaches a certain level. Therefore, there is a tendency to insert the flashboards in such a manner that they will never fail, thus permanently reducing spillway capacity, and increasing the possibility of dam failure by overtopping. This is what nearly happened at the Gore Mountain Dam at North Creek, NY. During the period of 1977-1980 NYSDEC operations personnel installed wide flange beams to support the wood flashboards. The approved design for the flashboard supports specified one inch diameter steel pins. However, operations personnel decided they would have less maintenance problems if they permanently secured the wood flashboards between six inch wide flange beams. Under this support the flashboards would never fail.

Around February 15, 1981, a sudden thaw and rain caused the water level at Gore Mountain Dam to rise within eight inches of the top of the dam. This level was about two feet, four inches over the top of the flashboards. The extra sturdy wide flange beam support system precluded any chance of flashboard failure. Fortunately, this abnormally high level was reported to the DEC by a local resident, who had observed it while snowmobiling. During the fall of 1981, DEC revised the flashboard support system so that the flashboards were properly supported by one inch diameter steel pins, which would fail in bending when the depth of flow over the top of flashboards reached one foot.

No. 2 Flashboard Failure Due to Ice and High Flows
Deferiet Project (Brookfield Power New York, 2000)

The Deferiet Hydroelectric facility is located on the Black River, NY. The dam consists of a 13 foot high Amburson spillway, 504 feet long, with 3 foot high wood flashboards and a 20 foot high sluice gate structure, 192 feet long. Each year the wood flashboards would be lost due, either to ice buildup behind the boards, or high river flows associated with a January thaw or spring runoff. Boards would not typically be reinstalled until after river flows subsided sufficiently to allow personnel to safely access the spillway. During the time the boards were lost, the plant operated with a reduced head pond elevation for 4-6 months each year, resulting in a reduced generating capacity of as much as 2 MW.

The owner's goal was to reduce the labor involved in maintaining the flashboard system, and to increase the total capacity and energy from the facility. Several alternatives were examined. They included replacing the fixed flashboard system with trippable wood flashboards, hydraulically actuated crest control gates, or inflatable flashboards. Inflatable flashboards, as shown in Figures 6.4-6 and 6.4-7, were chosen as the most economical option for the following reasons:

- The trippable wood flashboard option was highly susceptible to being destroyed each year, due to large debris and ice sheets being carried over the spillway.
- The hydraulically actuated crest gate option was discarded, due to its high cost and possibility of oil leaks within the river reach.
- Inflatable flashboards were considered, due to the relatively low installation cost and ease of operation.

The inflatable flashboard system consisted of individual rubber air bladders that are armored on the upstream side with 5/8 inch thick stainless steel plates. The total height of the system, when fully inflated, is 3 feet. The economics for this project provided for a 3 year payback period based solely on the increase in generation. The inflatable flashboard system offered other benefits. For example, part of the design criteria, mandated by the regulatory agencies, was the ability to pass 800 cfs for downstream walleye spawning. To accommodate this requirement, an independent 80 foot section of inflatable flashboards was incorporated into the design. This section could be raised or lowered independently from the remaining section of the rubber dam to pass these flows without lowering the entire impoundment.

Figure 6.4-6 Downstream Side of Rubber Dam (courtesy of Brookfield Power New York)

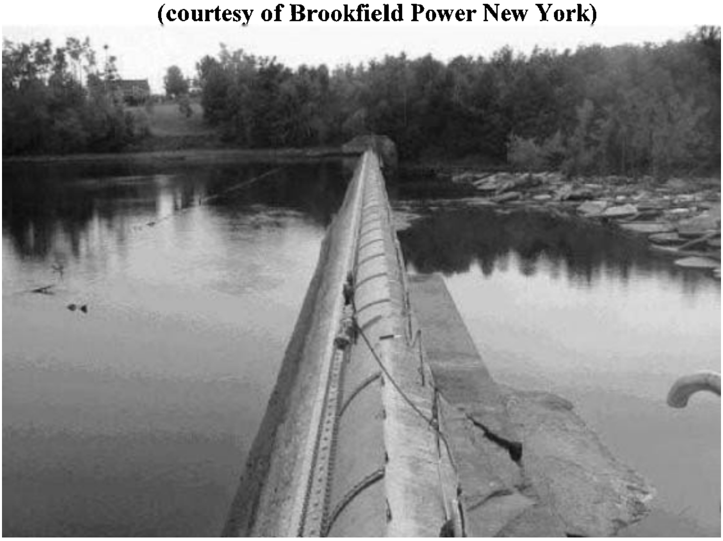


Figure 6.4-7 Upstream Side of Rubber Dam (courtesy of Brookfield Power New York)

No. 3 Personnel Safety, Lost Generation and O&M Costs Anson Project (Kleinschmidt, 1997)

The Anson Hydroelectric Project is located in Madison, ME. The 1929 vintage project generates 9 MW at 23 feet of head. The project is one of two that provides electric power and water to the owner's paper mill. The concrete gravity and stone masonry dam is 630 feet long and includes three sections of flashboard-regulated spillway totaling 528 linear feet. The flashboards are one-inch thick wood planks, 4.1 feet high, and supported vertically by 2-inch diameter steel pins (Figure 6.4-8). The project also included three deep gates used to sluice trash and debris from the entrance of the powerhouse forebay.

Figure 6.4-8 Anson Dam Original Flashboards Configuration (courtesy of Kleinschmidt)

The project's owner was concerned about the value of lost generation, the costs to maintain the flashboards, and the safety of the personnel who perform the flashboard maintenance. By varying the spacing of the flashboard pins, the flashboards were designed to fail in stages, but due to the impact on the flashboards by spillway flow, ice, and debris, the flashboards generally failed in a random pattern when overtopped by 1.5 to 2 feet. On average, the flashboards and pins were replaced three to five times a year, requiring 20 man-days per replacement. In addition to the labor required to replace the flashboards, additional labor was needed to minimize leakage through the flashboards, and keep them from free of ice. Fluctuation of the impoundment during the winter often resulted in the lifting of the flashboards due to the bonding of the ice sheet to the boards. This, then, resulted in increased leakage through the boards. Although an air bubbler was installed to reduce the flashboard lifting problem, leakage through the flashboards still led to ice build up on the downstream side. The ice build up was a condition that could prevent the flashboards from failing and would result in flooding upstream. The loss of the flashboards was estimated to prevent generation of an additional 5-10% over a current average annual generation of 52,200 MWH.

Further compounding the problem was the deterioration of the concrete along the spillway crest, resulting in an increase of leakage beneath the flashboards, and at times a loss of embedment support for the flashboard pins. In addition, alkali reactivity in the concrete had deteriorated the condition of the sluice gate structure to the point where only one of the three gates was operable, but with difficulty.

Environmental agencies were increasingly expressing their concern for the release of wood and other man-made materials into the waterways when the flashboards failed. Discussions had taken place related to the possible need to retrieve and remove the materials.

Over the years the owner had evaluated a number of options to improve control of spillway discharges. The owner had even evaluated constructing a new dam and raising the normal level of the impoundment by 3 feet. In 1995, due to the deteriorating condition of the dam's concrete structures, the owner undertook a project to repair them, improve control of spillway flows by replacing the flashboards, and allow the raising of the level of the impoundment by 1.5 feet when the project was relicensed.

The criteria used to evaluate the alternatives were: the economics of minimizing the use of civil works that required extensive and costly modifications; providing a spillway control system that would allow the raising of the normal level of the impoundment by 1.5 feet, while not increasing the potential for upstream flooding; use of a spillway control system that would minimize the use of materials or fluids that might create an environmental concern if released; and the ability to freely discharge large ice sheets and heavy floating debris.

Spillway control systems that were evaluated included crest gates (bottom-hinged, overflow gates), inflatable rubber dams, and various hinged flashboard arrangements with struts that would be manually released and reset. Tainter gates and vertical lift gates were not evaluated due to their need for significant civil works (gate piers and walkways), and potential for those civil works to obstruct the discharge of ice and debris. In addition, the civil works would reduce the spillway capacity of the dam unless significant modifications were made to lower the permanent crest of the spillway.

Limited construction drawings were available for the dam, because it had been rebuilt on more than one occasion since the hydroelectric potential at the site was developed in 1895. To support the proposed modifications and designs, a detailed topographic survey was performed of the dam site, along with limited geologic investigation to assess the capacity of the dam and foundation to sustain the increased hydrostatic loads.

In 1996 the existing gate structure was razed, a new structure constructed immediately downstream, and a single inflatable rubber gate 13.5 feet high by 40 feet wide was installed. In 1997-98, a single length of 5.6 foot high inflatable rubber dam was added across each of the spillways of the three sections of dam (Figure 6.4-9). The rubber gate and dam were selected over steel gates due to their simplicity of design, the ability to open (deflate) the gate without need of electric power, and the absence of materials or fluids that may create an environmental concern downstream. When deflated, the rubber dam also provided an unrestricted spillway capable of discharging any volume or size of debris or ice. The rubber dam also fitted well with the work of resurfacing the spillway dams to repair concrete deterioration, by not requiring any modification to the dam structure, as would be required to accommodate steel gates.

The inflatable rubber gate and the inflatable rubber dam on each section of the spillway can be operated separately, and allow the owner to maintain a constant impoundment level for flows up to 30,000 cfs, equivalent to the 3-year flow. For higher flows, the rubber gate and rubber dams are deflated and the crest level of the dam is the same as when flashboards were used.

Figure 6.4-9 Anson Dam Rubber Dam (courtesy of Kleinschmidt Associates)

The replacement of the flashboards resulted in an increase in power generation of 5-10% by increasing the amount of time that the project could operate at full head. The replacement also eliminated the labor and material costs that had previously been required to maintain the flashboards.

The reduction of the 4.1 foot fluctuations in headpond when the flashboards were out also benefited upstream wetlands, fish and wildlife habitat, and reduce shoreline erosion. The greater control over the level of the impoundment let regulating agencies agree to increasing the normal level of the impoundment 1.5 feet when the project was relicensed. The additional head provided an additional 7% increase in average annual energy.

No. 4 Insufficient Spillway Capacity and Loss of Flashboards Shelburne Dam (Kleinschmidt Associates, 1990)

The Shelburne Hydroelectric Project is located at the foot of the White Mountains, in Shelburne, NH. The 1915 vintage plant generates 3.1 MW at 3,100 cfs under a gross head of 17 ft. The project, along with five other hydroelectric projects, provides electric power to the owner's two paper mills.

The dam consisted of a 186 ft long section of 10 ft high vertical flashboards on a three to five foot high timber crib sill. Other discharge capacity was provided by four 6 ft wide sluice gates of varying height, and one 19 ft wide by 13.5 ft high log flume gate used as a sluice gate. The flashboards were tripped manually, using a cable winch to pull the support struts to control the release and minimize the loss of flashboards. The flashboards were divided into an upper and lower section, and each section was divided in two, allowing the flashboards to be tripped in increments of 25% (Figure 6.4-10).

Figure 6.4-10 Shelburne Dam Original Flashboard Configuration (courtesy of Kleinschmidt Associates)

The project had insufficient spillway capacity to routinely discharge the spring runoff or flashfloods associated with the intense summer thunder storms that occurred in the White Mountains, which would result in the loss of part or all of the flashboards. While the flashboards could sustain being overtopped by three feet, floating debris or ice would fall on the forest of flashboard support struts, which often caused uncontrolled failure of the flashboards. In addition, when the level of the impoundment rose more than three feet, flooding would occur along US Route 2, which ran along the impoundment.

The loss of even half of the upper section of the flashboards could result in more than a 15% reduction in generating head. In average years, the flashboard outages might reduce generation by 10 to 15 percent, but during wet years, when the river flows could not be controlled to allow the flashboards to be reinstalled, the generation could be reduced by more than 35%. After more than 40 years of use, the timber decking could no longer anchor and adequately support the flashboard struts, and flashboard failures were occurring more frequently. In addition, the timber crib sill was showing advanced deterioration and need of replacement, after more than 70 years of service.

Due to the need to rebuild the timber sill to maintain the flashboards and the impoundment, an internal decision was made to rebuild the dam and include means for increased control of the spillway discharge. The assessment and construction were to be completed within three years, and the project arrangement was to be selected on the basis of both capital investment and operating and maintenance costs.

Numerous studies for replacing the flashboards with various types of spillway gates had been performed since the mid-1930s. However, they all concluded that, while technically feasible, it was not economic to install gates, primarily due to the cost of cofferdams, and the inability to discharge sufficient flow when the dam was under construction. The new assessment used the results of the previous studies and evaluated the cost of replacing only a part of the existing flashboards with gates, and rebuilding the remainder of the dam using new flashboards.

The gates considered were hydraulically operated (bottom-hinged) crest gates, and an inflatable rubber dam. The flashboards considered would be either similar to the existing arrangement and release system, or inclined flashboards of a more permanent fashion. Due to the availability of past studies and subsurface investigation programs, additional investigations were not required, and only a detailed topographic survey of the dam site was performed to provide information to be used for both the assessment and the final design.

In 1990, 110 feet of the flashboard dam was replaced with three 25 foot wide by 10 foot high hydraulically operated crest gates. The remaining 76 feet of dam was rebuilt by replacing the timber crib sill with concrete, and installing new 10 foot high vertical flashboards that were similar to those currently used at the dam (Figure 6.4-11). Several changes occurred, which allowed the gates to be installed at this time but not in the past. These included the replacement of the four sluice gates and the large log flume gate in the previous six years to improve their operability and reliability for controlling the discharge of flows, especially during construction. In addition, the work of the cofferdam construction and the rebuilding of 76 feet of the flashboard dam was, this time, classified as maintenance work, rather than capital improvement. As a result of this classification, funding for a significant portion of the construction costs (40%) was obtained from maintenance budgets. The gate work was then economically feasible as only the work of the construction and installation of the three crest gates and supporting civil works was classified as capital improvement, which met the owners economic criteria.

Figure 6.4-11 Shelburne Dam New Crest Gates (courtesy of Kleinschmidt Associates)

Tainter and vertical lift gates were options that had been considered in previous studies, but the ability to open the bottom-hinged hydraulic crest gate without the need of any power, resulted in the crest gate being the preferred option for mechanical gates. The Obermeyer gate and an inflatable rubber spillway were also considered in the current assessment, but while providing all of the flexibility of the crest gate, along with being able to be installed along the full 186 feet of dam, they were not selected because (at that time in 1988) they were not widely used, nor were any in operation at the height required for the Shelburne Dam.

Beyond the increase in power generation and the reduction in the cost of labor and materials for replacing flashboards, there were two other significant environmental gains attributed to the installation of the gates. The first was that the gates provided a significantly finer control of the level of the impoundment, and nearly eliminated the large fluctuations in water level that occurred routinely, and often a number a times a year, when the flashboards were not in place. This resulted in a more stable environment for the significant wetlands that bordered the impoundment. The second was the project eliminated the release of tens of thousands of board feet of lumber into the river downstream of the dam whenever the flashboards failed or were released.

After a number of years of operation without loss or release of any flashboards, deterioration and weakening of the wood struts began to occur due to their exposure to a wet environment. While the new gates had nearly eliminated the loss of the flashboards due to high river flows, the flashboard struts were now subjected to deterioration because they were not replaced on a regular frequency. Routine maintenance and replacement of the struts now had to be performed, although at significantly lower costs than had occurred in the past.

No. 5 Over-Design of Flashboards and Failure of Spillway Crest Lower Pelzer Project (Kleinschmid Associates, 1996)

The Lower Pelzer Hydroelectric Project is located in Pelzer, SC. The 1895 project generates 3.3 MW at 1,500 cfs under a gross head of 40 feet. The project, in continuous operation since construction, was the first hydroelectric project in the United States to use overhead wires to transmit electricity over a long distance (3.3-kV, 3 miles).

The project's water retaining structures have a total length of 775 feet, and are stone masonry, gravity structures. The spillway dam is 351 feet long, with a 287 foot long spillway carrying 4 foot high, inclined flashboards. In addition to the spillway and turbines, the project can discharge flows through two 7 foot high by 9 foot wide sluice gates with mechanical operators.

Available construction drawings of the project indicated that the dam was to have been constructed 6 feet lower and without flashboards. At some unknown date, two feet of concrete and 4-foot high flashboards were added to the crest of the dam. The revised drawings indicated that the flashboards were inclined and supported by inclined struts. Over the years the design of the flashboards were further modified, and in 1995 the design, at the time of failure, looked as shown in Figure 6.4-12.

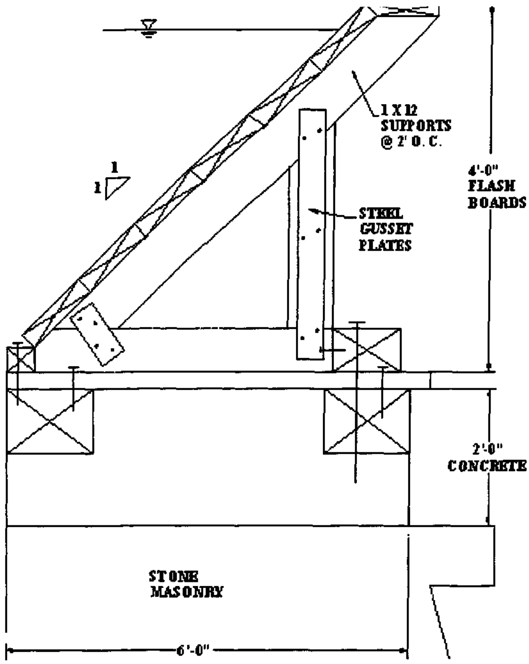


Figure 6.4-12 Pelzer Dam Original Flashboard Configuration (courtesy of Kleinschmidt)

In August 1995, a hurricane caused flooding on the river, resulting in the flashboards being overtopped by 5.5-foot. The flood had a return interval of 15-years and nearly equaled the 1973 flood of record. The flood was the third such event to reach the top of the project's flood walls.

The flashboards did not fail as would normally have been expected, but because of their current construction, remained intact, resulting in the loss of the top 7 feet of the dam's crest along 200 feet of length. The 11 foot loss of static head represented 90% of the submerged head on the turbines, and as a result, the turbines could not be operated.

In the design for the reconstruction of the dam, no other alternatives were considered for replacing the flashboards as a means to control the spillway capacity. Flashboards are an inexpensive way to increase head on a project while providing a reasonable spillway capacity, and the owner determined that the spillway would be rebuilt using flashboards, but with a means to control their release.

The spillway crest was reconstructed using concrete to replace the missing 7 feet of masonry. Because the project was considered to be historically significant, the downstream face of the new crest included a masonry facing of stone blocks anchored to the new concrete.

The new flashboards are also 4 foot high, and inclined using a wood strut system to maintain the flashboards in the up position (Figure 6.4-13).

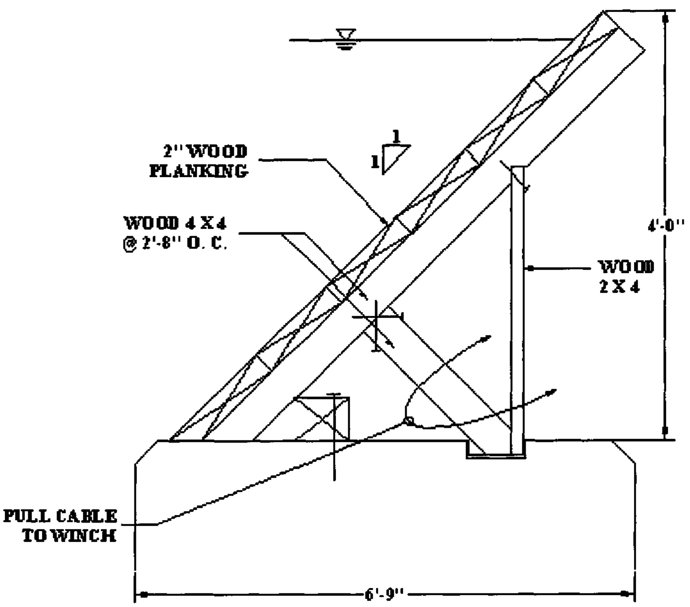


Figure 6.4-13 Pelzer Dam Redesigned Flashboard Configuration (courtesy of Kleinschmidt Associates)

The use of inclined boards provided a positive means for maintaining the flashboards in place without the need of mechanical connections. The flashboards were designed to withstand 5 feet of overtopping, at which point the inclined wood beam would fail. The flashboards can also be manually released using a cable to pull the struts free of the flashboards. The cable is connected to a 10,000 pound winch to pull the struts free, starting at the furthest strut from the winch.

The new flashboard design provides the owner with the ability to allow a controlled release of the flashboards at his discretion.

6.4.5 Collective Knowledge

1. Barrows, H.K. (1927). *Water Power Engineering,* 1st Edition. McGraw-Hill Book Company Inc., New York, NY.
2. Creager, W.P. & Justin, J.P. (1950). *Hydro-Electric Handbook,* 2nd Edition. John Wiley and Sons, New York, NY.
3. General Resources (manufacturers of pneumatic flashboard systems):
   - Obermeyer Hydro Inc., P.O. Box 668, Fort Collins, CO. 80522, USA
   - Bridgestone Industrial Products America, Inc., 155 West 72nd Street, Ste. 407, New York, NY. 10023, USA

6.4.6 Technical References

Brookfield Power New York. (2000). Project files, Brookfield Power New York, Liverpool, NY.

Creager, W.P. & Justin, J.P. (1950). *Hydro-Electric Handbook,* 2nd Edition. John Wiley and Sons, New York, NY.

Kleinschmidt Associates. (1997). Personal Communications re Madison Paper Anson Dam Rubber Dam, Kleinschmidt Associates, Pittsfield, ME.

Kleinschmidt Associates. (1996). Personal Communications re Consolidated Hydro Inc., Lower Pelzer Project, Kleinschmidt Associates, W. Columbia, SC.

Kleinschmidt Associates. (1990). Personal Communications re James River Shelburne Gate Project, Kleinschmidt Associates, Pittsfield, ME.

New York State Department of Environmental Conservation (NYSDEC). (1989). *Guidelines for Design of Dams*, NYSDEC, Albany, NY.

7.0 CHAPTER 7 – ANCILLARY SYSTEMS, SAFETY AND SECURITY

7.1 Introduction

Chapter 7 focuses on how ancillary systems, safety and security associated with civil works are identified, defined and implemented to extend the life of or upgrade, a hydroelectric project or dam. These project features and systems, integral to the operation of the hydroelectric facility include:

- Civil Works Systems – structural instrumentation, pressure relief, flood-proofing, and oil containment.
- Ancillary systems Critical for Emergency Operations –such as electric backup power, communications, and access.
- Recreational Project Safety.
- Project Security.

The purpose of Chapter 7 is to re-acquaint owners and other decision-makers with, and introduce engineers specializing in other disciplines to, the salient characteristics of these project features and systems. Failure of any of these systems can occur under non-emergency (e.g. routine operation or periodic testing), or emergency conditions. Because of the unique nature of each facility and the sensitivity of these systems to nuances of design and construction thereof, any project involving life extension and/or upgrading must, by definition, take into account these supporting systems.

The focus of the subsections in Chapter 7 differs from the discussion in previous chapters, in that they overview the critical nature of some project features, systems and subsystems which form part of, or are affected by, a life extension or rehabilitation project. References to pertinent resources for further information are contained in the text.

7.2 Civil Works Systems

This section overviews four systems associated with hydroelectric facilities: Structural Instrumentation; Pressure Relief; Flood-Proofing (principally of powerhouses, and; Oil Containment.

7.2.1 Structural Instrumentation

Instrumentation is designed and installed for the purpose of providing useful information on local or distributed properties for monitoring or surveillance related.

Monitoring and surveillance instrumentation for hydroelectric projects generally refers to Dam performance monitoring. Two references for instrumentation include:

- *Guidelines for Instrumentation and Measurements for Monitoring Dam Performance*, ASCE, 2000.
- *Engineering Guideline:* Section 9.4 (as updated), Federal Energy Regulatory Commission Division of Dam Safety and Inspections (D2SI), 2004.

Six types of structural measurements are commonly taken at hydroelectric projects, impoundment structures, and at relevant geologic features that can affect the projects:

- Pressure (barometric, pore, hydrostatic, dynamic, seepage, vacuum).
- Movement (regional, relative local, relative internal, relative free surface).
- Flow (seepage, leakage, integrated velocity).
- Temperature (thermal gradients, ambient air, ambient water, stratified reservoir, gallery, heat transfer).
- Stress (anchor tests, analog of strain).
- Strain (relative distortion of solids, anchor tests, analog of stress).

a) Problems

Problems with structural instrumentation generally arise from several sources including:

i. Failure.
ii. Interpretation.
iii. Inaccuracies and Repeatability.
iv. Incorrect Location.

i) Failure

Physical failure of an instrument, structurally, mechanically or electronically.

ii) Interpretation

Instrumentation is generally not read frequently enough or designed with enough precision to quantify background noise. As with electrical-measurement and communication interference, background noise herein is considered independent of the instrument (e.g. it is assumed that the instrument is working within is design limitations and accurately). Regional consolidations may explain trends observed when reviewing long-term readings. Often, this background noise goes undetected, or unrecognized, and therefore the interpretation of instruments is diminished considerably.

Most instrumentation is used to monitor thresholds. As long as the readings are below (e.g. piezometer) or above (e.g. load cell) a target, the theoretically conservative analysis of a structure's stability is sufficient to proclaim its integrity. Readings of instruments and assumed drivers (e.g. headwater and tailwater levels, flows, temperature) are attempted to establish correlations. When correlations are strong, confidence in the situation behind fluctuating readings grows. When the correlations are weak, confidence remains uncertain. If the purpose of an instrument is to reflect those conditions inherent in the assessment of the structure's integrity, and the readings fail to do so, then cannot be relied on the instrument.

iii) Inaccuracies and Repeatability

Problems can arise from poor installation, defective equipment, human error, or lack of calibration and maintenance. An instrument's failure to provide consistent data on a repeatable basis can sometimes be observed in electrical and mechanical devices. It might be argued that the instrument is inaccurate, but if the errant readings are intermittent, the problem is lack of repeatability. Therefore, an instrument reading can be accurate but not consistently accurate.

iv) Incorrect Location

While an instrument may be reading correctly it may not supply the right information if it is located incorrectly (e.g. not deep enough to measure a problem, such as intersecting a loose area or a failure surface).

b) Opportunities

Opportunities for improving or enhancing structural instrumentation may be identified by review of the following:

i. Examination of Ambient Conditions.
ii. Retirement of Instrumentation Programs.
iii. Instrumentation to Capture Unique Events and Long Term Behavior.

i) Examination of Ambient Conditions

Excavation drawings, construction photographs, records of test pits, exploratory borings and construction material removed provide an initial understanding of conditions. Often shelved to gather dust or lost entirely, excavation drawings and construction records might allow an analyst to determine how much theoretical stress relief has occurred in the geologic material left surrounding a hole, trench or tunnel, provided sufficient history and temporal variations in stress distribution is known about the undisturbed formation.

Since each hydroelectric project is unique, it is generally the case that pre-existing ambient conditions are not studied, other than those which the profession has established to be routine (e.g., exploratory borings, fault maps, test pits). In summary, to take advantage of instrumentation more comprehensively, a program to investigate and identify natural background noise permits an analyst to disaggregate those movements that occurred in foundations and abutments from the gross movements evidenced in the recordings.

ii) Retirement of Instrumentation Programs

An opportunity seized by some owners is to retire an instrumentation program because the readings show no significant trends or fluctuations. Arguments can be made that ongoing measurements pose risks to the feature being measured. Such is the case with post-tensioned tendons during lift-off testing and invar wire load assessments. Periodic exposure of anchor heads to corrosion and water fouling is a common justification.

When contemplating retirement, the original objective of the instrumentation should be considered. Instrumentation is installed to provide information about the performance of a structure and underlying supporting formations that was not known at the design and construction stages, but is theoretically useful in the operation stage. If, at the completion of design, features did not exist, then instrumentation would not be required since all movements, pressures, temperatures, stresses, and strains would be predictable. However, during operations, the reaction of the structures over time will be the best guide to the effectiveness of modifying or continuing an instrumentation program.

iii) Instrumentation to Capture Unique Events and Long Term Behavior

It may be productive, or opportunistic, to instrument a project for unprecedented events or for long term behaviors. With the emphasis now on performance-based surveillance, i.e. measuring parameters related to areas of risk or vulnerability, new instrumentation may need to be installed.

Examples include:

- Reading and maintaining seismographs and accelerometers attached to structures in zones of seismic activity.
- Capturing water-levels or pressures in dam and spillway galleries known to flood or suspected of flooding during high discharges, as an indicator to explain changes in piezometer readings and under-drain performance in the gallery.
- Capturing pressure fluctuations on spillway crests and at bucket surfaces and toes when spillway bays are discharging.
- Measuring tailwater levels at non-overflow sections, since tailwater may vary during extreme discharges and induce non-uniform erosion. Instrumentation and recording data during these unique events may enhance the understanding of the during rare, but serious, loading conditions.

Matrices of minimum recommended instrumentation for existing and proposed (new) structures have been promulgated by the Federal Energy Regulatory Commission's

Division of Dam Safety and Inspections (D2SI). Table 7.2-1 summarizes the recommendations for existing structures.

Table 7.2-1 Minimum Recommended Instrumentation for Existing Dams

MEASUREMENT TYPE	LOW-HAZARD POTENTIAL EXISTING DAMS	SIGNIFICANT & HIGH-HAZARD POTENTIAL EXISTING DAMS					
		Embankment	Concrete Gravity	Arch	Buttress	Non0Integral Spillway & Outlet	Integral Powerhouse
Visual	X	X	X	X	X	X	X
Reservoir		X	X	X	X	X	X
Tailwater		X	X	X	X	X	X
Drain Flow		X	X	X	X	X	X
Seepage Flow		X	X	X	X	X	X
Leakage Flow		X	X	X	X	X	X
Pore Pressure		X	X			X	X
Uplift Pressure		X	X			X	X
Surface Settlement							
Surface Alignment			X	X	X	X	X
Internal Movement							
Joint/Crack Displacement			X	X	X	X	X
Foundation Movement		X	X	X	X	X	X
Seismic Loads		X	X	X	X	X	X
Tendon Loads			X	X	X	X	X

(FERC D2SI Engineering Guidelines, Section 9-4)

7.2.2 Pressure Relief Systems

The use of the term 'dewatering' in this document refers to relieving and controlling fluid pressure (e.g. water and the associated atmosphere) that, left unchecked, could create instability of, or damage to an impoundment structure. The source of pressure is flow, leakage, or seepage of fluids, which can occur over large to microscopic solid surfaces. To control or relieve pressure is to control and relieve flow, seepage, and leakage on solid surfaces.

Two excellent sources for information on pressure relief systems are:

- *Design of Small Dams*, United States Department of the Interior, Bureau of Reclamation, United States Government Printing Office, 1974.

- *The Guide to Hydropower Mechanical Design,* ASME Power Technical Committee, HCI Publications, 1996.

Chapter 4 demonstrated that dams, powerhouses, spillways, and other impoundment structures, abutments, and foundations are all pressure-reducing or dissipation devices. Hydraulic potential drops across a project's structures, abutments, and foundations from reservoir level to tailwater, and between any two successive points on a gradient within them. The gradients or the distributed pattern of hydraulic potential surfaces within these structures, abutments, and foundations often remain enigmatic and therefore ill-defined. Instrumentation may have to be employed to better understand and define them. Control of hydraulic gradients can be provided by a pressure relief system.

Means to control pressure usually begins at the reconnaissance and design stage of a project. Elimination of natural water pathways in abutments and foundations is typically attempted with varied success by acknowledging that all pathways are impossible to identify and that those pathways left unsealed in earlier project phases can be identified and controlled once the project is in operation. Pre-construction works include cutoff curtains and keys. Post construction works include relief wells, grout curtains and slurry walls. The trend has been to investigate some of the more problematic pathways that are not sealed in the pre-construction phase, once the project is constructed. If problems do not arise, it is generally viewed that the elimination process was a success, at least for the time being. Many hydroelectric projects evolve into prototypical investigations and experiments in seepage and leakage control.

a) Problems

There are several problems with pressure relief systems. The manner used to control water movement and pressure, or potential buildup, is by diffuser or expansion device with separation (of water from contact with a surface) to atmospheric or sub-atmospheric pressure. Most drains used to dewater an impoundment structure are located in such places to intercept and divert *perceived* or *estimated* water-movement pathways. To segregate water from surfaces, diffusers must vent to ground potential and must dispose of influent under all inflow conditions. The ultimate drain or pathway ground may not be that which is identified as such on a drawing of a dam's cross-section.

Problems may arise because of decay of falsework, initially used to form and cast the under drains and left in place. Sometimes constructed of wood, the falsework becomes broken and splintered using conventional sewer-cleaning technologies (e.g. high-pressure jetting followed by vacuum extraction). Tools and debris dropped down risers into under drains while retrofitting air, water, and electrical conduits in galleries can also cause blockages.

Another problem observed is the inadvertent grouting of sections of underdrains, lift and block joint drains. This has been known to occur as a result of a consolidation and post-tensioning operation as contact underdrains after follow the bedrock profile creating sags to which suspended solids move and collect over time.

b) Opportunities

There are several corrective measures for pressure relief systems. If total seepage and leakage flows are known over the range of all expected net heads (potential differences), maintaining a solid ground is merely a matter of providing for extraction of the accumulations. Adequate extraction may not be provided by gravity-system capacities as envisioned by the original designers. Additional capacity would then be required. The solution may be a combination of flood-proofing measures combined with pumping systems.

7.2.3 Flood-Proofing

Hydroelectric facilities, intakes, powerhouses and the generation equipment, are particularly vulnerable to water encroachment and subsequent damage. Flood-proofing features incorporated in many existing hydroelectric facilities may remain dormant for long periods. Often installed during original construction or added after an event, these provisions may fall into disrepair through disuse. Infrequently implemented, often lacking automation, the effort to activate them in times of flood warning may require significant manpower. The feasibility of allocating human resources safely during a flood emergency is the first step in assessing if flood-proofing measures could be deployed with all the other tasks likely to be assigned to available staff.

Flood-proofing devices or measures to prevent flooding are a combination of mechanical, electrical, structural, and electronic devices. However, flood-proofing often extends beyond the confines of a facility, and awareness of the facility in relation to the total watercourse system is vital to ensure the survivability of the structure during extreme events. This awareness is normally included in the process used to develop the Emergency Action Plan and should be updated to recognize changes and alerts upstream and downstream of the facilities.

Flood-proofing prevents water from moving to locations of low potential, which are to be protected, from areas of higher potential. Two types of flood-proofing provisions exist: (1) leak-proof and (2) controlled leakage. The leak-proof provision is generally for residential applications where damage to exterior and interior finishes and furnishings can be as devastating as destruction of the entire structure. Controlled leakage provisions are more commonly found in powerhouses and appurtenances.

Most powerhouses and galleries are provided with floor drains or gutters. Both features typically intercept risers (vertical pipes usually embedded) that descend to one of the lowest levels in the entire project, the station drainage sump. The lowest level is usually the dewatering sump, which is used to un-water the station's draft tubes, penstocks, tunnels, shafts, intakes, and turbine chambers that cannot be drained to (are below) tailrace level. In small powerhouses, the drainage and dewatering sump may be the same. The fact that powerhouses and appurtenant structures are provided with gutters, drains, and sumps indicates a controlled leakage approach to design, operation, and maintenance under normal conditions.

Head and tailwater rating curves can be generated for most foreseeable conditions, but knowing which routes water will take from areas of higher to lower potential requires an understanding of the features that exist above normal head and tailwater levels. What may not be obvious are those features below normal head and tailwater levels that are close to failure is a flood event.

Leakage pathways are often unknown and unimportant. Leakage is controlled under normal conditions and conducted to the station drainage sump. Flooding is not a concern as long as the sump discharges by gravity, pumps, ejectors, or siphons. Most power stations use electric-motor-driven dry-pit or submersible pumps in sumps because station substructures contain equipment and storage galleries well below tailwater level.

Table C-5 (Appendix C) contains a summary of flood-proofing measures at powerhouses; a) under normal and, b) extreme flooding conditions.

a) Problems

The problems encountered with existing flood-proofing provisions and devices are generally associated with ignorance of extreme conditions, and neglect. Rarely used and often cumbersome, large, and difficult to access and operate, all these attributes may conspire to render the device ineffective or unusable during an emergency for which it was designed and installed.

Other problems include: deployment of sufficient manpower; accessibility of critical features; handling large gate panels in hurricane or tornado-level wind gusts; low-visibility problems; and temperature extremes. The practicality of flood-proofing has to be weighed in light of the emergency conditions expected.

Certain features such as immersed, embedded waterstops may have deteriorated over time and only extreme tail and headwater levels would indicate the evolved ineffectiveness. Gaskets and seals may deteriorate chemically and require replacement. Retrofits of flood doors and window bulkheads should include a re-analysis of uplift, slab and wall strength, and stability of the structure so fitted. Leakage through defective gaskets and waterstops may be tolerable if the sump pumps and station emergency generators are adequate for the task.

b) Opportunities

Understanding reservoir, conveyance, and tailwater hydraulics is the first step in assessing the benefits of and need for flood proofing. The development and updating of a flood protection plan for the site is an important process. To develop such a plan; a checklist could include items contained in Table C-5 (c) (see Appendix C).

7.2.4 Oil Containment Systems

Oil containment discussed herein includes the storage and control of oil in hydroelectric mechanical and electrical systems that are use oil for lubrication, power, and cooling. It also includes spills and discharges potentially arriving in a station or auxiliary unit sump. All who have inspected powerhouse facilities have occasionally observed absorbent socks or mats, expansion pellets, holding tanks, skimmers, separators, and other flotation devices, along with centrifuges. Because of the infrequent occurrence of inadvertent spills, batch processing generally prevails, which requires holding or containment tanks sufficient to exceed in volume any spill that might occur.

Mechanical and electrical systems that commonly contain oil include:

- Turbine Governor.
- Gate Operator (Hydraulic).
- Turbine and Generator Shaft-Bearing Lubrication and Cooling.
- Powerhouse Elevator.
- Turbine Shift-Ring Pistons.
- Valve Operator (Hydraulic).
- Hydraulic Torque Devices.
- Lubrication of Auxiliary Equipment Bearings.
- Cooling of Auxiliary Equipment Bearings.
- Instrumentation of Critical Equipment Features (Temperature, Solids).
- Maintenance Vehicles and Equipment.
- Oil-Fired Heating—Boilers and Unit Heaters.
- Dielectric in Oil-Filled Breakers.
- Dielectric in Transformers.
- Closed-Circuit Cooling Working Fluid.

a) Problems

Oil containment in hydroelectric facilities is dictated by three key regulations and compliance requirements:

1. Spill Prevention Control & Countermeasure (SPCC) plans required by U.S. EPA regulation 40 CFR Part 112.

Current regulations state that an SPCC plan is required at facilities that have (among other items not relevant to the site): above ground oil storage totaling 1,320 gallons. Containers with capacities of 55 gallons and greater are included to make the 1,320-gallon determination. Tanks and electrical transformers are also included. The plans must address, among other items, secondary containment around all oil-handing operations and storage, security, employee training, spill response, and record keeping. SPCC plans are usually prepared by a professional engineer.

Within structures, secondary containment can be a relatively straightforward provision. However, containment around outdoor electrical equipment requires significant engineering attention. Once prepared, the SPCC plan will require construction of secondary containment systems around the oil containment. A variance to the secondary requirement can only be given if it is not practical to construct such containment. Cost is not an allowable consideration in determining practicality.

2. State Pollution Discharge Elimination System (SPDES) permit.

   SPDES permits are issued by a state's department of environmental protection and delegated under the federal National Pollution Discharge Elimination System (NPDES) to monitor discharges into state (public) waters.

   If a facility discharges water that flows to interior floor drains, to an oil-water separator and then to a natural watercourse, this type of discharge often requires an SPDES permit. Some SPDES permits also require identification of storm water discharges, and are likely to include an effluent limitation, and a routine sampling requirement from the building sump/oil-water separator to demonstrate compliance with that limitation.

3. Petroleum Bulk Storage (PBS) regulations, if applicable.

   Current Petroleum Bulk Storage regulations generally apply if total storage in tanks exceeds 1,100 gallons. Unlike the SPCC regulation, this does not include drums and transformers.

Containment includes de-contamination and disposal provisions. Hazmat handlers should be posted in the area. Besides eyewash stations, respirators, and rubber gloves, non-absorbent chemically resistant clothing may be required, depending on the labeling of materials used.

Potential contamination exists in the water-borne influx to the spillway and powerhouse intake in the form of societal discards of partially filled cans, barrels, and other containers that hold toxic materials. Unfortunately, rivers do still act as sewers and provisions must be made for disposal of the type of waste that appears in the form of debris at trash racks and spillway gates.

A well-operated and maintained facility will have MSDS (Material Safety Data Sheet) data on all regulated and restricted-use chemicals. Many large powerhouses have packaged wastewater treatment plants for human waste processing and disposal. These plants require storage of disinfectants such as chlorine compounds. Chemicals associated with the package treatment plant add to the list of regulated chemicals.

Direct immersion transformers and circuit breakers require containment in substations to prevent contamination of the surroundings. Standby power sources that include emergency generator (EG) sets require safe fuel storage. Safety of design, construction, and operation of these facilities are codified by such institutions as the NFPA and are generally regulated by the Environmental Protection Agency through delegation of authority to qualified state agencies. The EPA retains oversight responsibility.

7.3 Systems Critical for Emergency Operations

While somewhat beyond the scope of life extension and upgrading of civil works, this sub-section covers three important ancillary systems that relate to overall performance improvement of powerhouse facilities. These include Backup Electrical Power; Communications; and Access. Failure of any of these features can occur under non-emergency (e.g. routine operation or periodic testing) or emergency conditions. Because of the unique nature of each facility, and the sensitivity of these features to nuances of design and construction, consideration of these systems in any life extension and upgrade project is part of a successful undertaking.

Ancillary systems can be grouped either by how they are furnished and installed, or by function. In addition, some ancillary systems are critical to energy production and some are not. The difference between a critical and non-critical system and the interrelationships between the systems, are as follows:

Critical Systems – Rather than attempt to generalize which ancillary systems may be critical to such a diverse population of hydroelectric facilities, the engineer may determine the critical and non-critical systems by reviewing the start-stop sequence or ladder logic diagrams for operation of the turbine-generator equipment. Learning the permissives, those precursor conditions that must be met prior to, and during, the start-up sequence for generation, provides an overview of critical elements. For hydroelectric facilities with black start capability, review of the starting sequence may save time. This procedure would list only those systems necessary to start the units from a non-spinning state without station service.

Non-Critical Systems – These systems may or may not be included in an annunciation sub-panel. For example, a station drainage sump may have a high-water alarm which is not interlocked with control circuitry for the turbine-generator set. A high differential pressure on a manually operated duplex strainer that is in series with a cooling water circuit, may simply require a valve adjustment to the clean basket. Failure of a spillway gate operator, such as shearing of a shear key on a drive shaft, would not trigger a unit shut down at most facilities. However, such a failure could be critical in a dam safety context.

Interrelationships – Master controls that govern subservient systems are common in the turbine-generator unit sequence. Large turbine components (runner, stator, rotor, shaft sections, couplings and bearings) could be the subservient system, while, ironically, a relatively small, inexpensive RTD, could on the other hand, be part of a master system. This control relationship can also exist in substations, and with other auxiliary equipment that drives the generation function.

7.3.1 Electrical Backup Power (EBP)

Hydroelectric generating stations, including dam and headwater facilities, have requirements for electric service. This requirement includes powering loads ranging from high horsepower motors to instrumentation and control systems. The nature of a power plant will require power to both AC and DC loads. Electrical backup power (abbreviated "EPB" herein) is a source of energy to loads considered vital or critical to hydroelectric plant and flow-control operations when isolated from transmission and distribution (station-service) grids.

The age, size, physical location, climate (extreme temperatures), ownership (utility or IPP), and complexity of the existing, or soon to be designed hydroelectric station, all impact the analysis required to properly account for backup power requirements. Access to existing utility facilities may serve to provide for backup power under certain design scenarios, but the most likely worst case scenario would involve blackout conditions.

It may seem obvious, but determining the scenario(s) that must have power provided is an important first step in beginning the design. The goal of the analysis needs to be understood. Are you effecting a safe shutdown of the station and awaiting the grid to be restored, or is a blackstart the objective? Is maintaining a certain flow a requirement? Is the station manned or controlled remotely? An understanding of manpower staffing and response times, and procedures will help with this effort. Finally an understanding of the plant loads, their power requirements, operating times, and the scenario duration will be necessary to determine the load profile that for supplying backup power. While it is easy to focus on the just the equipment, the needs of plant personnel must be considered as well, i.e. lights, heat.

Design Considerations

The design of the EBP may involve modernizing or upgrading existing equipment, or in some cases may be a first time effort. With modern communication and control equipment, the EBP requirements today will differ from those of a past design. The first step is to determine the type and size of loads and develop a load profile. The loads can be AC, single and three phase, and DC, battery powered or powered from inverter equipment. The design of the DC system may have already had a load profile developed when sizing the battery, but any change in the nature of the plant loads, (i.e. additions of modern control equipment, relays, SCADA) may have altered an initial load profile. With deregulation of the power grid, outage durations may have to be reconsidered as well, due to the fact that the utility no longer owns the power plant and does not have the same dedication or commitment to repowering the facility as they once had. For some facilities, they even may be part of master utility or ISO plans for grid blackstart, and this impact needs to be taken into consideration.

Once the load profile has been determined, selection of the backup power source can move forward. Along with this effort, the requirements to connect this generation to existing load centers, powerboards, or single power units must be determined. This "backup generator" may be a backup genset powered by diesel, propane, natural gas or the plans may involve water-powered generation. Automatic start and auto-transfer schemes can be used with installed backup generation. It may be desirable for some installations; i.e. small hydro or multiple unit stations, to self generate. This would be dependent on such things as historical water availability. However as long as governor pressure can be maintained for a time interval long enough to have dispatched crews restart the unit, this option is available for minimal effort. However, provisions can be made to have a smaller gasoline powered generator used to power a small section of station service to feed small loads necessary for a restart. Transfer switches could be installed on the governor pump feeds to afford direct connections. Procedures would have to be developed to ensure safe operation of equipment in these type situations. No matter which generation source is ultimately picked, isolation from the grid is most likely necessary and protective schemes need to be in place to prevent undesirable equipment damage.

Loads

Inherently obvious, but nonetheless important is having an understanding of the loads that need serving by the EBP system. Plant loads could be considered critical or non-critical, but they can be categorized by basic equipment functions including lubrication, sealing, cooling, water control, protection, drainage, flow control, bypass flow, instrumentation, power supplies, HVAC, gates and valves, and communications. Table 7.3-1 summarizes a checklist of basic hydro facility loads that are usually connected to station service and EBP needs

Designs for a new installation may be able to group the necessary loads on a critical bus that can be fed from the EBP through a transfer mechanism. This can be automatic transfer switches or breakers (contactors) controlled by an appropriate protection and control scheme. This may not be as easy in an older facility where all the loads may be controlled from a single powerboard or MCC or not grouped in a critical/non critical fashion. Dam gates and valves may not even derive their power from the powerhouse station service and may be fed from utility distribution. Grounding issues need to be taken into consideration. Code and utility requirements may need to be examined in facilities that have been divested from utilities where grand fathered installations were used with good utility practice. The use of batteries for control and emergency loads requires the proper selection relative to size, construction (calcium, selenium, or antimony or even non acid filled batteries) location, and room temperature. The design of a DC system will be at desired voltage level and the availability of equipment to operate at certain voltages, particularly 250 VDC, is becoming harder to find. Protection and coordination issues with fuses or molded case breakers are different with DC than AC.

Table 7.3-1 Hydroelectric Station Service Loads Supplied by EBPs

	EQUIPMENT FUNCTION	EQUIPMENT LOADS
Control	Bypass Flow	Bypass Flow Control:
Control	Communications	Maintaining external communications; radio, SCADA, and internal PLC unit control
Control	Control of Water	Watering and Dewatering: Pumps; Motorized Valves; Control/Monitoring Systems
Control	Drainage	AC & DC Pump Motors; Motorized Valves; Control/Monitoring Systems
Control	Gates and Valves	Gate and Valve Operators:
Control	Sealing	Supply (open-circuit) and Circulation (closed-circuit) Pumps; Drainage Pumps; Motorized Strainers; Motorized Valves; Control/Monitoring Systems
Control	Unit Flow Control	Generation Flow Control: velocity transducers and transmitters; HPU pumps
Power	Cooling	Drainage Pumps; Motorized Strainers; Circulation Pumps; Motorized Valves; Control/Monitoring Systems
Power	Ground Fault Protection	Control of Stray and Ground-Fault Currents
Power	HVAC	Fan motors; motorized louvers; thermostats
Power	Instrumentation	Monitoring: exciting amperage;
Power	Lubricant Processing	Centrifuge; Micro-Strainer
Power	Lubrication	Supply Pumps; Circulation Pumps; Motorized Valves; Control/Monitoring Systems
Power	Power Supply	Equipment Power Supply: grounded, single and 3-phase outlets; 120/208VAC distribution panel boards
Power	Water Processing	Pump motors; mixer motors; instrument-air dryers and compressors;

Prime Mover

The selection of the prime mover and hence the fuel storage requirements is an important consideration. The prime mover could be a water turbine, but more than likely will be a reciprocating engine of some type and cost, durability, and longevity are issues in selection.

EBP systems can be stationary or mobile. A mobile diesel engine backup unit could be designed to serve more than one facility. A truck-mounted generator or crane is an example of a mobile EBP in that it is used in an emergency as a backup to stationary power supply systems. However, access and adequate structural support must be considered prior to mobilizing such devices in an emergency, if they are indeed available at all. Sometimes heavily counterweighted and fitted with outriggers for stability as a function of pick-distance and load, a supporting structure must be able to withstand concentrated loads from the outrigger pads.

Stationary EBPs, those EBPs that include a prime mover and likely a fuel storage requirement have additional considerations. The prime mover could be a water turbine, but more than likely will be a reciprocating engine of some type and cost, durability, and longevity are issues in selection. Thus the fuel could be diesel, kerosene, LPG, propane, natural gas, or even gasoline, and its transport could be via pipeline with storage or trucked in and stored in tanks. Storage tanks pose their own inherent cautionary considerations including location and access particularly during high water events and environmental restrictions or impacts related to spill prevention and countermeasures. Also, the amount of storage would need to be determined based on anticipated outage times. Maintenance and periodic testing of the backup power supply needs to be considered, as well as the electrical interconnection and isolation to allow for maintenance. More than likely, a backup generator will require its own battery and charger and will have its own maintenance issues.

General

Maintenance procedures, and system documentation all need to be in place to ensure that the EBP is operated to fulfill its design function. The final requirement is testing and than can range from individual device test and calibration to a full simulation of a loss of power to test both equipment and personnel response to a loss of power or blackout incident.

a) Problems

Problems with electrical backup systems generally occur in three areas: (1) inadequate sizing of loads to power supply or changes in station configuration; (2) problems with fuel storage and (3) maintenance and upkeep of gensets.

Inadequate sizing and changes in configuration of EBPs is often remedied by tracking and documenting the inter-relationships between EBP and critical or vital loads.

Periodic field verification of those loads connected to the EBP will provide an enhanced understanding and prevention of difficulties in emergency situations. By maintaining up-to-date wiring diagrams; wiring schedules; bus schedules; conduit, duct, tray, and chase schedules; equipment terminal-bar connection schedules; and equipment schematics, the sizing of the station EBP can be verified.

Some electric loads are known to be oversized traditionally. Gate-hoist motors are typical of this category. Selected to move gates at resistances far in excess of sliding seal friction, roller-bearing frictional torques, weight of gate, dynamic fluid drag forces, and hoist losses, the motors can be drawing increasingly more amps from an MG or EG set than originally anticipated. Verifying that the load needed to move critical gates during an emergency can be adequately provided by the EBP system and is a worthwhile periodic inspection and test.

Problems associated with fuel storage generally relate to contamination. Periodic starting and stopping of a diesel engine supplied by a "day tank" is a good practice but may not mimic an actual duty cycle and lead to a false sense of security, since short-term exercises may hide the fact that sludge or water is accumulating in the main storage tank that feeds the day tank. Supplying appropriate sensors to detect contaminants in or periodic sampling of the tank may be prudent.

Another source of problems is the testing and upkeep of the condition of the prime mover(s). Torque tests and routine maintenance should be included in the overall program for the station's prime movers. The condition of any gearbox and clutch assembly associated with the prime mover's torque transfer requires monitoring. What ultimately is moved include a generator shaft and field rotor, motor armature, gears, wire rope, chains, pulleys and sheaves, and finally a gravity, dynamic or friction load of some type.

b) Opportunities

To anticipate and avoid the problems noted above, there are several opportunities to verify the condition and reliability of EBPs.

- It is recommended that the load versus power source verification be done periodically to check applicability.
- Maintenance and testing of the EBPs on a regular basis ensures verification of reliability.

Review of emergency operating scenarios and reliance on EBP under a variety of emergency events—flooding, snow, loss of grid or communication, will improve the reliability of EBP systems.

Some excellent references for Electrical Backup Power include:

- Clemen, David M. (1999). *Hydro Plant Electrical Systems*. HCI Publications, Inc., Kansas City, MO.
- ASME Hydro Power Technical Committee. (1996). *The Guide to Hydropower Mechanical Design*. HCI Publications, Inc., Kansas City, MO.

7.3.2 Communications

Communications in the context of the hydroelectric facility include person-to-person, machine-to-person, person-to-machine, and machine-to-machine. The format includes audio (voice), alphanumeric text (hand-writing), digital, and analog signals associated with hydropower plant operation and control. Most communications are electromagnetic in nature and require power supplies at both transmitting and receiving terminals. The function of communication is to relay relevant and reliable information from source to receiver for monitoring and control.

The trend to automate began with hard-wired control devices such as switches and relays that would impose decisions and actions without human interference. Instruments usually initiate component-to-component communications that are implicit to control systems. An instrument is generally composed to two parts, a transducer and a transmitter. As an example, the transmitter may simply be an electrical circuit, the current of which varies with change in temperature in an element within the circuit that acts as the transducer. Not coincidentally, an instrument used to control and monitor electrical, mechanical, and structural features of a hydroelectric project has a history behind its nomenclature. Most communication begins with vibration, the strings of a viola or harpsichord and the reed of a clarinet, for example. Electronic instruments communicate with high-frequency electro-magnetic vibrations called oscillations.

a) Problems

Problems arising in communications stem from the two component elements: source, message, decoding or translation, and transmitting medium and receiver.

Source, message decoding, or translation – The problem of reliability is often associated with the source. The human voice is unlimited in its ingenious variations and manifestations. Background noise and bad acoustics often suggest that the intercom system needs to be upgraded to improve communication. However the operator communication skills and the equipment are both important to generate the source.

Transmitting medium and receiver – Problems arise in transmitting acoustic information in air, primarily from interference. Interference is due largely to conflicting and extraneous wave generation and transmission that is difficult to filter out from the sounds of interest. Background noise caused by operating equipment is one source of contamination, unless a specific piece of equipment has a signature sound that is varying substantially. In this case, such a sound can be considered a predictive maintenance issue. Most control rooms in powerhouses are enclosed in acoustically insulated spaces to permit quiet working conditions. Here again, being experienced or attuned to ambient sounds of an operating facility under normal conditions is important. Aberrant background noise can alert the operators to developing problems.

b) Opportunities

Frequency bands operating within the locale of a hydroelectric facility may include the following:

- AM radio – 535 kilohertz to 1.7 megahertz.
- Short wave radio – bands from 5.9 megahertz to 26.1 megahertz.
- Citizens band (CB) radio – 26.96 megahertz to 27.41 megahertz.
- Television stations – 54 to 88 megahertz for channels 2 through 6.
- FM radio – 88 megahertz to 108 megahertz.
- Television stations – 174 to 220 megahertz for channels 7 through 13.
- Garage door openers – alarm systems – Around 40 megahertz.
- Standard cordless phones – Bands from 40 to 50 megahertz.
- Baby monitors – 49 megahertz.
- Radio-controlled airplanes – 72 megahertz.
- Radio-controlled cars – Around 75 megahertz.
- Wildlife tracking collars – 215 to 220 megahertz.
- Cell phones – 824 to 849 megahertz.
- New 900 MHz cordless phones.
- Air traffic control radar – 960 to 1,215 megahertz.
- GPS – 1,227 and 1,575 megahertz.
- Deep-space radio communications – 2,290 megahertz to 2,300 megahertz.

Communications in and around hydroplants are inherently noisy.

As a point of reference, telephone (hardwired) communications are restricted to a bandwidth of about 400 to 3,400 Hz, or well below the AM radio bandwidth.

This partial list is an indication of the electromagnetically "noisy" environments in which hydroelectric facilities operator. Source strength and location obviate most interference generated by the above examples. Local, variable-frequency discharges to ground and spurious high-frequency noise exists in most hydroelectric facilities. One common generator test, the telephone influence factor test, is an example.

Several technical opportunities exist for improving communications including wireless technology, fiberoptics, and video.

Fiber-optic transmission of electromagnetic waves is another method of controlling ambient noise. Shielded control and instrumentation wire operating at low voltage and ampacity is another, provided the shielding is properly grounded. Hands-free radio headsets should be tested under all foreseeable ambient conditions to determine the most reliable make and manufacturer. A greater distribution of high-resolution video cameras with 360° field of vision and zooming capabilities follows the adage, "a picture is worth a thousand words."

7.3.3 Access During Emergencies

The following observations are based on practical advice from hundreds of inspections of dams and hydroelectric facilities. This general subject includes all methods of access from a non-project location to project features.

Planning emergency access includes evaluating the features incorporated into the project. The project feature that may require access during emergencies could be remote or at project site. Remote examples include instruments, such as stream gauges, off site gate or valve operators, surge tanks and transmission line towers. On site project features include air and water lines in galleries, draft-tube piezometers, station sumps, penstock air-release and vacuum valves and standby generators.

A four step process to evaluate access includes access conditions, means of access equipment to bring, and consideration of consequences if access is not achieved.

1. Evaluate the access conditions including investigating the probability of reaching the project remote location under all emergency conditions. Consideration include highways, bridge, and airport closures and the effort required in terms of personnel and time to arrive at a non-project location.
2. Evaluate the means of access the project features may include automobiles, trucks and vans, ambulances, elevators, hiking with backpacks, dirt bikes, cable and harness man-lifts, helicopters, horses, bicycles (used on long spillway decks and single tracks), shielded cable and conduit, long and chaotic boat rides, snowmobiles, an all-terrain vehicles (ATV), rope rappelling, ladders, spiral stairways, manhole rungs, and waders, to name a few. A reasonable degree of physical fitness may be required of the person accessing the feature, depending on its location and nature. Deployment of staff to remote locations may have to factor in physical fitness considerations. Confined-space approaches to items of interest may require SCBA (self-contained breathing apparatus), or stationary and portable fans. Underwater access may require use of SCUBA (self-contained underwater breathing apparatus).
3. Evaluate equipment to bring. Equipment and tools that are required during any foreseeable emergency may not be available and may require transport from another location. Remote communications may enable staff to dispatch the requisite items from a remote location. If communication does not exist and automation fails, then a scout may be sent with a radio or cellular phone for reconnaissance to apprise others of what has happened and what is required.

4. Consider the possibility and consequences should conditions prohibit access entirely. Inability to access project features may be included in an Emergency Action Plan as potential failure modes of a structure.

Sunny-day emergencies have different access challenges than those occurring under severe weather conditions. What may be routine and simple operations in good weather may be physically impossible under severe weather conditions.
In formulating contingency plans for improving emergency access it may be useful to:

- Review construction drawings, studies, reports, and photographs of the features of concern.
- Review state, county and local road-closing and traffic-restriction laws and regulations.
- Review local airport-closing laws and restrictions.
- Review labor, OSHA, and other agreements and laws regarding employee rights under life-threatening conditions.

Other measures to improve contingencies for access include:

- Inventory and pre-contract for outsourced services, especially during emergencies.
- Consider providing some emergency provisions on site. Improved weather forecasting might be one way to address the access dilemma regarding remotely operated, unmanned stations.
- Stocking adequate spare parts for critical pieces of equipment is another corrective measure to consider.

7.4 Recreational Project Safety

With increasing pressure for additional features and multi-purpose uses of hydroelectric facilities and environs, recreation has always been a serious issue for project owners. Public safety can be defined as the protection of the public from the inherent risks associated with hydroelectric facility operation, through the use of barriers, guides, restricted areas and other institutional protective devices.

Often this infrastructure is affected by a life extension and upgrade project. Excellent references on this project feature are contained in:

- Federal Energy Regulatory Commission (FERC). (1992). *Guidelines for Public Safety at Hydropower Projects.* FERC, Washington, DC.
- Federal Energy Regulatory Commission (FERC). (2001). *Safety Signage at Hydropower Projects.* FERC, Washington, DC.

The function of pubic safety devices is to protect the public from the inherent risks: deep, cold water, turbulence, unforeseen fluctuations in water level, unseen discharges (overflow dams), increased currents and eddies, unstable banks, steep side channels (in tailraces) and a multitude of other hazards associated with an operating hydroelectric facility. This includes the application of public safety measures such as boat barriers at reservoirs, buoys at special hazards, low bridge warning signs, canoe/kayak portage signs and trails, warning signs at project hazardous areas, nighttime lighting, and fences around substations and other areas.

The key problems associated with public safety are striking a balance between allowable access and what is safe. Fishing is always better where it is most dangerous and no matter what protective guards are in place, the risk remains. What practical measures can an owner take to protect the public?

The FERC manuals are a useful guide to identifying project hazards and applying safety devices and measures to enhance public safety. These include sections on:

- Education and Information.
- Warning Devices.
- Restraining Devices.
- Escape Devices.
- Project Operating Procedures Review.

Figure 7.4-1 shows a hydropower dam with typical public safety devices.

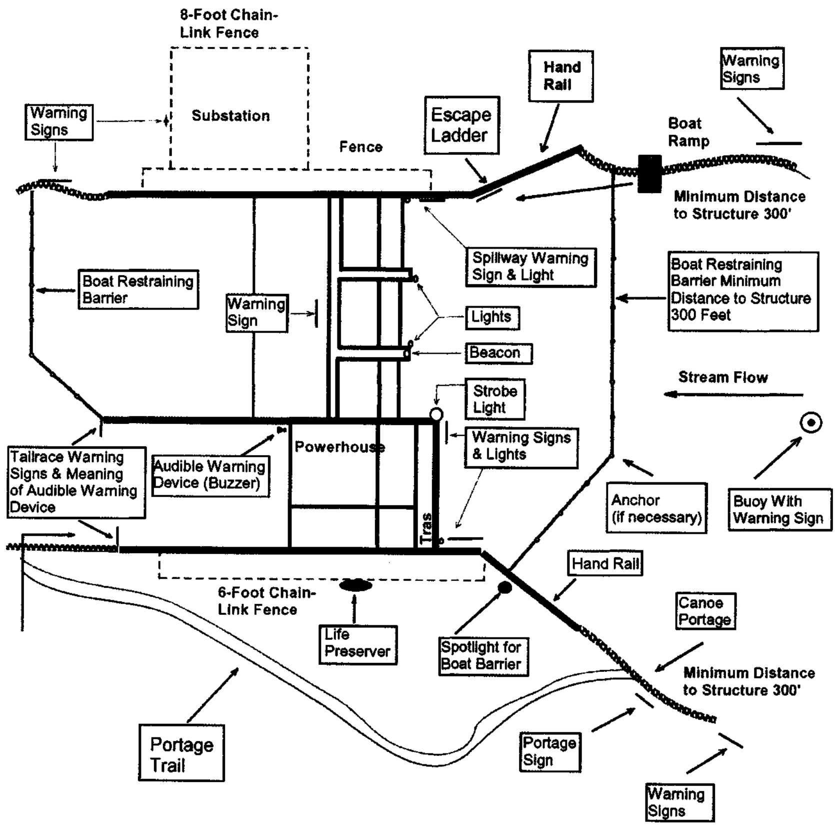


Figure 7.4-1 Typical Location of Public Safety Devices (FERC, October 2001)

7.5 Project Security

This section presents a limited and general discussion of hydroelectric facility security and system concepts. The discussion is limited to what an owner and operations staff can and must now consider in terms of restricting access, detection, and notification (e.g., interacting with law-enforcement agencies).

a) Problems

While hydroelectric project owners and operators were always diligent about public safety, there was always a 'gray' area where allowing the public near the facilities, e.g. fishing from the tailrace platform, compromised project security and safety. Often, in the interest of good community practice and pressure from NGOs, the tailrace was open for fishing, and owners tolerated the risks.

On September 11, 2001, the rules changed. Hydroelectric owners and operators must now consider both structural, and non- structural, responses to project security.

This is clearly a rapidly evolving area of engineering practice, and applications to the hydroelectric industry will likely continue to evolve. Several important references for the practitioner include:

1. *Structural Design for Physical Security – State of the Practice.* ASCE Task Committee, 1999.

   This document contains very specific design methods for understanding load definitions (such as ballistic and explosive attacks and forced entry), and the structural responses (such as analysis of structural components; security window, door, and other utility raceway design; and retrofitting of existing structures). Moreover, Appendix A to the book provides a very comprehensive *Threat Determination Guide*, developed by the USACE Protective Design Center for Expertise, Omaha NE. The guide systematically walks the reader through a facility threat assessment, as a starting point for plant security design or retrofit.

2. The Federal Energy Regulatory Commission (FERC) Hydropower Security Program, 2003.

   Following 9/11, FERC instituted a program to conduct a four-step security review of projects in its jurisdiction. This program includes:

   - Security Assessments – an evaluation of the current state and appropriateness of the on-site security system and what needs to be done at a project, or facility, to address concerns regarding security.
   - Vulnerability Assessments – addresses the four key factors:
     - Identifies the weak spots or vulnerable project features.
     - Assesses the potential threat to the facility based on history and security incidents.
     - Addresses the consequences of such an attack.
     - Addresses the effectiveness of the security system to counter such an attack.
   - Security Plan – a document that characterizes the response to security concerns at a project or facility that may include structural and non-structural responses and features as well as company procedures to follow based on changing threat conditions or situations.
   - Integration of Security Concerns with Emergency Action Plans.

3. Interagency Forum on Infrastructure Protection (IFIP)

   The IFIP group was started in 1997 to address "security" type issues at Federal projects. The committee consisted mainly of Federal government agencies, USBR, BPA, USACE, TVA, and FEMA but also has members from several laboratories, such as Sandia National Laboratory and Lawrence Livermore National Lab. IFIP has developed a *Risk Assessment Methodology for Dams* (RAM-D) that can help evaluate sites from a security perspective.

b) Opportunities

The function of project security is protection of project property and operation from hazardous threats both in the sense of malicious but not project threatening vandalism, which is the predominant issue at hydroelectric facilities, and the overall threat of terrorism at a larger and more calculated scale. The evolving thinking in this area is in terms of project security strategy:

- Threat identification and prioritization.
- Protection measures in place.
- Mitigation or enhancement of existing features.
- Response – are your plans and communications in place?
- Recovery – have you thought through each scenario?

Developing both structural and non-structural responses to these issues is key to enhancing project security, not only in the wake of the 9/11 physical attacks, but enhancing security from all threats, from vandalism to cyber-attacks.

Structural responses include such things as retrofitting structures with security doors and windows. Non-structural responses include changes and updates in operating or emergency procedures.

The following list is not comprehensive but presents some guidance on opportunities for improving security at hydroelectric facilities.

- Disable rarely used equipment, which if used irresponsibly could create significant consequences. An example would be removal and safe storage of shear keys used to lock radial gears on gate hoist shafts. Many machined elements such as these must be provided by the manufacturer, or be available in excess as spare parts. The delay involved in diagnosing an ineffectual spinning shaft until the missing key is discovered, and then the attempt to locate the key could possibly enable a response team to arrive and neutralize a threat.
- Limit access to a facility to a few individuals and ensure that control-related information is only available to authorized personnel.
- Secure observation wells and other instrument shafts to deter placement of explosives over time until a large number of them are packed for detonation.
- Secure heel drains in gravity dams interconnected with risers to gallery gutters because these could offer a means of secreting explosives until detonation. Even if dam failure is not achieved, compromise of cutoffs, increased uplift, prolonged forensic investigations, lowering of reservoirs and associated costs would likely be extremely high.

- Disable a long gated spillway by temporarily removing sections of rails upon which traveling hoists roll, or otherwise securing hoists during non-critical flow periods.
- Secure valuable references off site, where possible, and store those on site securely, including parts lists of all critical ancillary equipment. Determine which equipment parts could be expeditiously removed and secured in the event of anticipated misuse.

The following non-structural corrective measures were suggested in October 2001 in *Hydro Plant Security Measures – A Draft Laundry List of Suggested Security Measures For Dam Owners and Operators* prepared by various hydroelectric owners and operators.

At Manned Sites:

- Categorize/prioritize structures by potential impact – (i.e., loss of life, loss of critical services, extensive property damage, etc.). Focus efforts on those structures which have serious impacts should failure occur.
- Assess vulnerabilities. Identify critical components of the project that could result in large, uncontrolled releases of water (spillway gates, penstocks, valves, etc.).
- Consider development of security plans that cover normal operations and various states of emergency alerts.
- Review and update Emergency Action Plans to make sure telephone numbers are correct and support personnel are familiar with the EAP.
- Talk to employees about the potential threats. Have plant staff be on the look-out for unusual activities or personnel on project sites. Call police or supervisory personnel to inform them of any unusual activities. It should be emphasized that plant staff should not confront individuals or attempt to detain them.
- Consider performing a self audit of the project security features such as fences, gates, alarm systems, and early warning devices. Check functionality of existing early detection systems such as intrusion alarms, headwater alarms, and tailwater alarms.
- Consider increasing frequency of walk downs of project areas.
- Restrict unauthorized access to sites. Make sure normally locked gates are locked. Make sure automatic gates function properly.
- Consider restricting access to public areas near the dam if necessary for security.
- Work with law enforcement personnel to patrol and enforce no boating areas near dam.
- Consider limiting large trucks from driving over dams.
- Review black start procedures if applicable to the project.

At Remote Sites:

- Consider installation of intrusion alarms and cameras at unmanned sites.
- Work with local law enforcement to increase drive-by surveillance.

- Consider fly-overs of project facilities to supplement surveillance.
- Consider staffing high risk, remotely located projects, or consider implementation of a roving surveillance crew.

Retrofitting hydroelectric facilities with security devices can be a costly issue, but one that an owner must unfortunately accept as a cost of doing business. Security practitioners warn that throwing money at the problem, guns, gates, and guards, isn't necessarily the best plan. Instead, promoting a security response strategy can identify areas where money should be spent for the greatest impact.

In a recent study by the USACE, in connection with its RAM-D program security improvements at hydroelectric facilities were grouped into several general areas and generic cost estimates developed for budgetary purposes.

7.6 References

ASCE Task Committee. (1999). *Structural Design for Physical Security – State of the Practice.*

ASCE. (2000). *Guidelines for Instrumentation and Measurements for Monitoring Dam Performance.* ASCE, New York, NY.

ASME Hydro Power Technical Committee. (1996). *The Guide to Hydropower Mechanical Design.* HCI Publications, Kansas City, MO.

Clemen, David M. (1999). *Hydro Plant Electrical Systems.* HCI Publications, Kansas City, MO.

Federal Energy Regulatory Commission (FERC). Division of Dam Safety and Inspections (D2SI). (n.d.). *Engineering Guideline: Section 9.4* (as updated). FERC, Washington, DC.

Federal Energy Regulatory Commission (FERC). (1992). *Guidelines for Public Safety at Hydropower Projects.* FERC, Washington, DC.

Federal Energy Regulatory Commission (FERC). (2001). *Safety Signage at Hydropower Projects.* FERC, Washington, DC.

United States Department of the Interior, United States Government Printing Office. (1974). *Design of Small Dams.*

APPENDIX A

HYDROELECTRIC AND DAM ENGINEERING

LIBRARY OF GENERAL RESOURCES

In addition to the many direct references contained in the bibliography and text, the following is a compilation of useful hydroelectric and dam engineering resources.

TABLE OF CONTENTS

A.1 GENERAL HYDROELECTRIC DESIGN AND OPERATION

1. Zipparro, V. & Hasen, H. (1993). *Davis' Handbook on Hydraulics*, 4th Edition. McGraw-Hill, Inc., New York, NY.
2. Creager, W.P. & Justin, J.P.. (1950). *Hydro-Electric Handbook*, 2nd Edition. John Wiley and Sons, New York, NY.
3. Gulliver, J.S. & R.E.A. Arndt. (eds.) *Hydropower Engineering Handbook*, McGraw-Hill, Inc., New York NY.
4. Barrows, H.K. (1927). *Water Power Engineering*, McGraw-Hill, Inc., New York, NY.
5. Linsley, Ray K. & Franzini, Ray K. (1972). *Water Resources Engineering*, McGraw-Hill, Inc., New York, NY.

A.2 AMERICAN SOCIETY OF CIVIL ENGINEERS (ASCE)

Website: www.asce.org

The American Society of Civil Engineers publishes many technical references on Hydropower and Dams, through the Energy Division- Hydropower Task Committees. Some recent publications of note include:

1. *ASCE/EPRI Civil Engineering Guidelines for Planning and Designing Hydroelectric Developments*, Volumes 1-4, 1989
2. *Alternatives for Overtopping Protection of Dams*, 1994
3. *Guidelines for Instrumentation and Measurements for Monitoring Dam Performance*, 2000
4. *Guidelines for Inspection and Monitoring of In-service Penstocks*, 1998
5. *Guidelines for Evaluating Aging Penstocks*, 1995
6. *Guidelines for Design of Intakes for Hydroelectric Plants*, 1995
7. *Guidelines for Rehabilitation of Civil Works of Hydroelectric Plants*, 1992
8. *Hydraulic Design of Spillways*, 1995
9. *Lessons Learned from the Design, Construction and Operation of Hydroelectric Facilities*, 1994
10. *Steel Penstocks,* [Manual of Practice No. 79], 1993

A.3 ASSOCIATION OF STATE DAM SAFETY OFFICIALS (ASDSO)

Website: www.asdso.org

A national non-profit organization of state and federal dam safety regulators, dam owners and operators, engineering consultants, manufacturers and suppliers, academia, contractors and others interested in dam safety. Our vision is to lead the US dam safety community with a strong, unified voice and effective programs and policies toward the furtherance of dam safety.

ASDSO Publications
Association of State Dam Safety Officials
December 2000

Suggested Reference Materials for State Dam Safety Programs

In an effort to help improve state dam safety programs, the Association of State Dam Safety
Officials Model Library Committee identified and recommended a core collection of materials on dam safety. These references were compiled in a publication dated December 2000.

ASDSO's Suggested Reference Materials for State Dam Safety Programs include approximately seventy-five references from sixteen general categories of materials, all of which represent knowledge deemed essential for state dam safety personnel. The categories comprise a cross-section of technical, classical and historical documents.

Categories

1. General Handbooks, Reference Manuals, Textbooks
2. Dam Performance, Incidents, and Historic Failures (Includes Case Histories, Analyses/Statistical Evaluations, Lessons Learned, Legal Cases, Hearings, Books)
3. Inspections, Operating Procedures and Maintenance
4. Hazard Classification, Inundation Studies, and Emergency Preparedness
5. Monitoring, Instrumentation, and Surveillance
6. Dam Design and Analysis (a) General (b) Earthen Dams and Tailings Dams (c) Concrete Gravity Dams (d) Arch and Buttress Dams
7. Construction Inspection and Materials Testing
8. Spillway Capacity Evaluation, Modification, and Hydraulics
9. Site Exploration, Foundations, Geology, Geotechnical
10. Seismic Evaluation and Design Earthquake Selection

11. Hydrology, Design Flood and PMF
12. Overtopping Evaluation and Protection
13. Seepage and Piping
14. Laws, Regulations, and Guidelines
15. Dam Safety Programs/Management
16. Risk Assessment

A.4 ELECTRIC POWER RESEARCH INSTITUTE (EPRI)

Website: www.epri.com

The Electric Power Research Institute (EPRI) has conducted unique research for the hydroelectric industry. This research, sponsored by its members--and available to the public for a fee--has included many important topics for the hydroelectric industry.

a) Hydropower Modernization Guidelines

Through a joint collaborative arrangement, this ASCE Task Committee Report is being developed cooperatively with the EPRI Hydropower and Renewables Program as Volume 6 of their Hydropower Modernization Guideline seven-volume set:

1. TR 112350 – V1: *Guideline for Modernization of Hydro Plants Vol. 1: Overview*, 1999.
2. TR 112350 – V2: *Guideline for Modernization of Hydro Plants Vol. 2: Hydromechanical Equipment*, 2000.
3. TR-112350 – V3: *Hydro Life Extension Modernization Guides Vol. 3: Electromechanical Equipment*, 2001.
4. TR-112350 – V4: *Hydro Life Extension Modernization Guides Vol. 4 and 5: Auxiliary Mechanical and Electrical Systems*, 2001.
5. TR 112350 – V7: *Guideline for Modernization of Hydro Plants Vol. 7: Protection Controls and Automation*, 2000.

b) Hydropower Technology Round-Up Report Series

1. TR-113584 – Vol. 1: *Using Environmental Solutions to Lubrication*
2. TR-113584 – Vol. 2: *Rehabilitating and Upgrading Hydro Plants*
3. TR-113584 – Vol. 3: *Steel Penstock Coating and Lining Rehabilitation*
4. TR-113584 – Vol. 4: *Hydropower Technology Roundup Report: Accommodating*

Wear and Tear Effects on Hydroelectric Facilities Operating to Provide Ancillary Services

5. TR-113584 – Vol. 5: *Flow Measurement at Hydro Facilities: Achieving Efficiency, Compliance, and Optimal Operation*
6. TR-113584 – Vol. 6: *Hydropower Technology Roundup Report: Directions for Hydro Research and Development*
7. TR-113584 – Vol. 7: *Remotely Operated Vehicle (ROV) Technology: Applications and Advancements at Hydro Facilities*
8. TR-113584 – Vol. 8: *Meeting Hydro Staffing Challenges: Tools for Workforce Development and Improvement*

c) Other Reports

1. TR-114008 – *A Scoping Study of Sediment Management Issues at Hydroelectric Facilities*. Prepared by EA Engineering, Science & Technology, Inc. 2000.
2. TR-112013 – *Catalog of Assessment Methods for Evaluating the Effects of Power Plant Operations on Aquatic Communities*. Prepared by EA Engineering, Science & Technology, Inc. 1999.
3. TR-111517 – *Review of Downstream Fish Passage and Protection Technology Evaluations and Effectiveness*. Prepared by Alden Research Laboratory, Inc. 1998.
4. TR-104120 – *Fish Protection/Passage Technologies Evaluated by EPRI and Guidelines for their Application*. Prepared by Stone & Webster Engineering Corporation, [RP2694-1], 1994.
5. TR-100345 – *Uplift Pressures, Shear Strengths, and Tensile Strengths for Stability Analysis of Concrete Gravity Dams*. Volume 1. [RP2917-5], 1992.
6. TR-100345 – *Concrete Gravity Dam Strength Database*
7. GS-6365 – *Guidelines for Drilling and Testing Core Samples at Concrete Gravity Dams*. Prepared by Stone & Webster Engineering Corporation. [RP2917-5], 1989.
8. AP-5273 – *Design Guidelines for Pressure Tunnels and Shafts*. Prepared by University of California at Berkeley. [RP1745-17 (see 7/87 for Volume 2 and 4/91 for Volume 3)], 1987.
9. AP-4715 – *Roller-Compacted Concrete for Dams*. Prepared by Morrison-Knudsen Engineers, Inc. [RP1745-16], 1986.
10. AP-4714 – *Inspection and Performance Evaluation of Dams, Guide for Managers, Engineers, and Operators*. Prepared by Morrison-Knudsen Engineers, Inc. [RP1745-14], 1986.
11. MERLIN – *Analysis of Leesville Dam Report*, [1000165], Published June, 2000

12. CRFLOOD – *A Numerical Model to Estimate Uplift Pressure Distribution in Cracks in Concrete Gravity Dams*: *Volume 4*, [TR-101671-V4], November 1992.
13. *Investigation of Uplift Pressures and Shear and Tensile Strengths for Concrete Gravity Dams*, [GS-7100], December 1990.
14. *Design Guidelines for Pressure Tunnels and Shafts*, [AP-5273], June 1987

A.5 FEDERAL ENERGY REGULATORY COMMISSION (FERC)

Website: www.ferc.gov

The Federal Energy Regulatory Commission (FERC) regulates both the construction and operational phase of a project. Dam safety is a critical part of the Commission's hydropower program and receives top priority. Before projects are constructed, the Commission staff reviews and approves the designs, plans, and specifications of dams, powerhouses, and other structures. During construction, Commission staff engineers frequently inspect a project, and once construction is complete, Commission engineers continue to inspect it on a regular basis.

1. *Division of Dam Safety and Inspections Operating Manual* (updated biannually)
2. *Engineering Guidelines for the Evaluation of Hydropower Projects* (updated as needed)
3. *Guidelines for Public Safety at Hydropower Projects* – March 1992
4. *Dam Safety Performance Monitoring Program (DSPMP)/Potential Failure Modes Analysis* – April 2003
5. *Safety Signage at Hydropower Projects*- October 2001

A.6 INDUSTRY PUBLICATIONS

Website: www.hcipubs.com

HCI Publications
410 Archiblad Street
Kansas City, MO 64111
Phone: (816) 931-1311

In addition to *Hydro Review* magazine, *Hydro Wire, HRW* and *Hydro Alert*, HCI has published several key references as follows:

1. American Society of Mechanical Engineers (ASME). Power Technical Committee. (1996). *The Guide to Hydropower Mechanical Design*, HCI Publications, Kansas City, MO.

2. Clemen, David M. (1999). *Hydro Plant Electrical Systems*, HCI Publications, Kansas City, MO.

3. *HydroReview (Annual) Industry Source Book.* HCI Publications, Kansas City, MO.

Other industry periodicals include:

1. *International Water Power & Dam Construction,* IWPDC, Middlesex, UK.
2. *The International Journal on Hydropower & Dams,* Aqua-Media, Surrey, UK.

A.7 US ARMY CORPS OF ENGINEERS (USACE)

Website: www.usace.army.mil

Over the years, the USACE has published a series of USACE Civil Works Engineer Manuals of importance to the hydroelectric industry. Some selected titles include:

a) Hydropower

EM 1110-2-1701: *Hydropower*, 1985.
EM 1110-2-3001: *Planning and Design of Hydroelectric Power Plant Structures*, 1995.
EM 1110-2-4205: *Hydroelectric Power Plants Mechanical Design,* Ch. 1., 31 Jul 1996.
EM 1110-2-3006: *Hydroelectric Power Plants Electrical Design*, 1994.

b) Concrete

EM 1110-2-2000: *Standard Practice for Concrete for Civil Works Structures*, 2001.
EM 1110-2-2006: *Roller-Compacted Concrete*, 2000.
EM 1110-2-2002: *Evaluation and Repair of Concrete Structures*, 1995.
EM 1110-2-2102: *Waterstops and Other Preformed Joint Materials for Civil Works Structures*. 1995.
EM 1110-2-2704: *Cathodic Protection Systems for Civil Works Structures*, 1999.
EM 1110-2-6050: *Response Spectra and Seismic Analysis for Concrete Hydraulic Structures*, 1999.
EM 1110-2-6054: *Inspection, Evaluation and Repair of Hydraulic Steel Structures*, 2001.

c) Hydraulic and Mechanical

EM 1110-2-1411: *Standard Project Flood Determinations* [ENG BUL 52-8], 1965.

EM 1110-2-1419: *Hydrologic Engineering Requirements for Flood Damage Reduction Studies*, 1995.
EM 1110-2-1420: *Hydrologic Engineering Requirements for Reservoirs*, 1997.
EM 1110-2-1424: *Lubricant and Hydraulic Fluids*, 1999.
EM 1110-2-1612: *Ice Engineering*, 1999.
EM 1110-2-1619: *Risk-Based Analysis for Flood Damage Reduction Studies*, 1996.
EM 1110-2-2701: *Vertical Lift Gates*, 1997.
EM 1110-2-2702: *Design of Spillway Tainter Gates*, 2000.
EM 1110-2-3200: *Wire Rope Selection Criteria for Gate-Operating Devices*, 1998.

d) Dams

EM 1110-2-1602: *Hydraulic Design of Reservoir Outlet Works*, 1980.
EM 1110-2-1902: *Stability of Earth and Rock Fill Dams*, Ch. 1., 1970.
EM 1110-2-1908: *Instrumentation of Embankment Dams and Levees*, 1995.
EM 1110-2-2105: *Design of Hydraulic Steel Structures*, 1994.
EM 1110-2-2201: *Arch Dam Design*, 1994.
EM 1110-2-2200: *Gravity Dam Design*, 1995.
EM 1110-2-2300: *Earth & Rock-Fill Dams General Design & Construction Considerations*, 1994.
EM 1110-2-2400: *Structural Design of Spillways & Outlet Works*, 1964.

e) Tunnels and Conduits

EM 1110-2-1603: *Hydraulic Design of Spillways*, 1990.
EM 1110-2-2005: *Standard Practice for Shotcrete*, 1993.
EM 1110-2-2104: *Strength Design for Reinforced - Concrete Hydraulic Structures*, 1992.
EM 1110-2-2901: *Tunnels and Shafts in Rock*, 1997.
EM 1110-2-2902: *Conduits, Culverts and Pipes*, 1997.
EM 1110-1-2907: *Rock Reinforcement*, 1980.
EM 1110-1-3500: *Chemical Grouting*, 1995.
EM 1110-2-3506: *Grouting Technology*, 1984.
EM 1110-2-4300: *Instrumentation for Concrete Structures*, 1987.

f) Rehabilitation Technology

The US Army Corps of Engineers, Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program was initiated in 1984 and was brought to completion in 1998. Technology developed under this 14-year effort focused on seven problem areas: concrete and steel, geotechnical, hydraulic, electrical and mechanical, environmental, coastal, and operations management. The primary goal of this research effort was to develop affordable technology that would extend the service life of the nation's aging infrastructure. Technology transfer is available in the forms of bulletins, technical notes, material data sheets, technical reports, and videos. To locate information about technology developed under this program, search the website. The following subsections are an abbreviated listing of REMR Reports relevant to hydroelectric rehabilitation and life extension.

g) Concrete and Steel Structures

1. REMR-CS-1: *Engineering Condition Survey of Concrete in Service*, by Richard L. Stowe, and Henry T. Thornton, Jr. [AD-A148 893]
2. REMR-CS-3: *Latex Admixtures for Portland Cement Concrete and Mortar*", by Dennis L. Bean and Tony B. Husbands [AD-A171 352]
3. REMR-CS-5: *Instrumentation Automation for Concrete Structures*
4. REMR-CS-6: *In Situ Repair of Deteriorated Concrete in Hydraulic Structures: Feasibility Studies*, by Ronald P. Webster and Lawrence E. Kukacka [AD-A182 297]
5. REMR-CS-9: *Inspection of the Engineering Condition of Underwater Concrete Structures,* by Sandor Popovics and Willie E. McDonald [AD-A208 295]
6. REMR-CS-16: *Repair of Dam Intake Structures and Conduits: Case Histories*, by Roy L. Campbell, Sr., and Dennis L. Bean [AD-A192 819]
7. REMR-CS-43: *Structural Evaluation of Riveted Spillway Gates*, by John E. Bower, Mark R. Kaczinski, Zouzhang Ma,Yi Zhou, John D. Wood, and Ben T. Yen [AD-A281 492]
8. REMR-CS-45: *Detection of Structural Damage on Miter Gates*, by Brett C. Commander, Jeff X. Schulz, and George G. Goble [AD-A285 248]
9. REMR-CS-46: *Repair and Maintenance of Masonry Structures: Case Histories*, by Edward F. O'Neil [AD-A294 186]
10. REMR-CS-47: *Performance Criteria for Concrete Repair Materials, Phase I*, by Peter H. Emmons and Alexander M. Vaysburd [AD-A295 136]
11. REMR-CS-48: *Evaluation of Injection Materials for the Repair of Deep Cracks in Concrete Structures*, Paul D. Krauss, John M. Scanlon, and Margaret A. Hanson [AD-A302 262]
12. REMR-CS-50: *A Conceptual Design for Underwater Installation of Geomembrane Systems on Concrete Hydraulic Structures*, by J. Chris Christensen, Matthew A. Marcy, Alberto M. Scuero, and Gabriella L. Vaschetti [AD-A304 491]
13. REMR-CS-53: *Applications of Roller-Compacted Concrete in Rehabilitation and Replacement of Hydraulic Structures*, by James E. McDonald and Nancy F. Curtis [AD-A326 634]
14. REMR-CS-54: *Evaluating the Stability of Existing Massive Concrete Gravity Structures Founded on Rock*, by Robert M. Ebeling, Michael E. Pace, and Ernest E. Morrison, Jr. [AD-A329 714]

15. REMR-CS-55: *Evaluation of Criteria for Uniformity of Roller-Compacted Concrete*, by Brian H. Green, Billy D. Neeley, and Toy S. Poole [AD-A341 119]
16. REMR-CS-56: *Evaluation of Grouting Materials for Anchor Embedments in Hardened Concrete*, by Willie E. McDonald [AD-A337 585]
17. REMR-CS-621: *Performance Criteria for Concrete Repair Materials, Phase II Summary Report*, by Alexander M. Vaysburd, James E. McDonald, Randall W. Poston, and Keith E. Kesner [AD-A362 896]
18. REMR-CS-631: *Repair and Rehabilitation of Dams: Case Studies*, by James E. McDonald and Nancy F. Curtis [AD-A372 898]

h) Electrical and Mechanical Applications

1. REMR-EM-4: *Hydroelectric Generator and Generator-Motor Insulation Tests*, by Robert H. Bruck and Ray G. McCormack [AD-A212 924]
2. REMR-EM-5: *Lubricants for Hydraulic Structures*, by Ward B. Clifton and Alfred D. Beitelman [AD-A213 260]
3. REMR-EM-6: *Mechanical Properties and Corrosion Behavior of Stainless Steels for Locks, Dams, and Hydroelectric Plant Applications*, by Ashok Kumar, Ali A. Odeh, and J. R. Myers [AD-A219 490]
4. REMR-EM-71: *High-Solids and 100-Percent Solids Coatings: A State-of-the-Art Investigation*, by John Baker and Alfred D. Beitelman [AD A247 557]

i) Geotechnical Applications

1. REMR-GT-2: *Improvement of Liquefiable Foundation Conditions Beneath Existing Structures*, by Richard H. Ledbetter [AD-A160 695]
2. REMR-GT-3: *Geotechnical Aspects of Rock Erosion in Emergency Spillway Channels*
3. REMR-GT-8: *Review of Consolidation Grouting of Rock Masses and Methods for Evaluation*, by R. Morgan Dickinson [AD-A198 209]
4. REMR-GT-15: *Plastic Concrete Cutoff Walls for Earth Dams*, by Thomas W. Kahl, Joseph L. Kauschinger, and Edward B. Perry [AD-A234 566]
5. REMR-GT-17: *Applications and Testing of Resin-Grouted Rockbolts*, by Timothy S. Avery and James E. Friant [AD-A245 980]
6. REMR-GT-19: *DAMSEAL—An Expert System for Evaluating Dam Seepage*, by Roger L. King and Wendell O. Miller [AD-B171 367]
7. REMR-GT-20: *Evaluation of Overturning Analysis for Concrete Structures on Rock Foundations*, Shannon and Wilson [AD-A273 485]

8. REMR-GT-22: *The State of Practice for Determining the Stability of Existing Concrete Gravity Dams Founded on Rock*, by James K. Meisenheimer [AD-A298 577]

j) Hydraulics Applications

1. REMR-HY-2: *Floating Debris Control; A Literature Review*, by Roscoe E. Perham [AD-A184 033]
2. REMR-HY-3: *Elements of Floating Debris Control Systems*, by Roscoe E. Perham [AD-A200 454]
3. REMR-HY-4: *Icing Problems at Corps Projects*, by F. Donald Haynes, Robert Haehnel, and Leonard Zabilansky [AD-A266 343]
4. REMR-HY-5: *Ice Control Techniques for Corps Projects*, by F. Donald Haynes, Robert Haehnel, Charles Clark, and Leonard Zabilansky [AD-A301 541]

A.8 US BUREAU OF RECLAMATION (USBR)

Website: www.usbr.gov

The Bureau of Reclamation, with 14,000 MW of hydropower is an excellent resource for information. Some selected titles include:

1. *Linings for Irrigation Canals*, 1963.
2. *Canals and Related Structures* [Design Standard 3].
3. *Design of Small Canal Structures*, 1974.
4. *Design of Small Dams*, 1974 (revised 1987).
5. FIST Manuals.

Reclamation maintains a series of manuals entitled *Facilities Instructions, Standards, & Techniques* (FIST) which pertain to the operation and maintenance of hydroelectric equipment. These manuals are maintained on the web in Adobe format and can be used by hydroelectric operators as sources of information and training. A partial list of FIST manuals is attached below.

- Volume 2 - Mechanical Maintenance:

  2-1 *Alignment of Vertical Shaft Hydro Units*, Revised Fall 2000

  2-2 *Field Balancing Large Rotating Machinery*, January 1983, Reprinted 1994

  2-3 *Mechanical Governors for Hydraulic Units*, July 2002

2-4 *Lubrication of Powerplant Equipment*, November 1990

2-5 *Turbine Repair*, August 1989

2-6 *Protective Coatings for Water Passage Surfaces of Turbine Scrollcases, Turbine Parts, Draft Tubes*, n.d.

2-7 *Mechanical Overhaul Procedures for Hydroelectric Units*, July 1994

2-8 *Inspection of Steel Penstocks and Pressure Conduits*, September 1996

2-9 *Inspection of Unfired Pressure Vessels*, August 2001

6. Operation & Maintenance Programs, including:

   - Generator insulation program
   - Doble program
   - Voltage regulator/excitation system alignment program
   - Electronic governor alignment program
   - Mechanical governor alignment program
   - System stability responsibilities and WSCC (Western Systems Coordinating Council) support
   - Automation assistance program (efficiency and technical support)
   - Modular SCADA (supervisory control and data acquisition) program and development effort
   - Electrical Power Review of O&M program
   - Mechanical Power Review of O&M program
   - Plant grounding, safety, and EMF (electromagnetic fields) program
   - Diagnostics and field test support program
   - Penstock inspection program
   - Mechanical program support (alignment, bearing, vibration, and turbine repair support)
   - Pressure vessel inspection program

A.9 UNITED STATES SOCIETY ON DAMS (USSD)

Website: www.ussd.org

Technical Reports and Miscellaneous Publications

1. *Aging of Dam Foundations*, (2001)
2. *Annotated Bibliography on Roller-Compacted Concrete Dams*, (1994)
3. *Anthology of Dam Modifications Case Histories*, (1996)

4. *Bibliography on Performance of Dams During Earthquakes*, (1984)
5. *Bibliography on Reservoir-Induced Seismicity*, (1986)
6. *Compilation of U.S. Dams with Strong Motion Instruments and Reservoir Seismicity Networks*, (1985)
7. *Construction Testing of Embankment Materials Containing Large Particles*, (1988)
8. *Current U.S. Practices for Numerical Analysis Of Dams*, (1985)
9. *Dam Modifications to Improve Performance During Strong Earthquakes*, (2003)
10. *Dam Construction Project Management Guidelines*, (2002)
11. *Development of Dam Engineering in the United States*, (1988)
12. *Directory of Computer Programs in Use for Dam Engineering in the United States*, (1992)
13. *General Considerations Applicable to Performance Monitoring of Dams*, (1986)
14. *General Guidelines for Automated Performance Monitoring of Dams*, (2002)
15. *Guidelines for Earthquake Design and Evaluation of Structures Appurtenant to Dams*, (1995)
16. *Guidelines on Design Features of Dams to Effectively Resist Seismic Ground Motion*, (2003)
17. *Guidelines for Inspection of Dams After Earthquakes*, (2003)
18. *Improving Reliability of Spillway Gates*, (2002)
19. *Key References for Hydraulic Design*, (1991)
20. *Observed Performance of Dams During Earthquakes*, Volume II (2000), Print Version OR CD-ROM Version
21. *Observed Performance of Dams During Earthquakes*, Volume II (2000), Print Version AND CD-ROM Version
22. *Planning Processes for the Development of Dams and Reservoirs*, (2003)
23. *Proceedings – Fifth Benchmark Workshop on Numerical Analysis of Dams*, (1999)
24. *Proceedings of the Earth Summit Workshop*, (1993)
25. *RCC Construction for Dam Rehabilitation*, (2003)
26. *Reservoir Triggered Seismicity*, (1997)
27. *Strong Motion Instruments at Dams - Guidelines for Their Selection, Installation, Operation, and Maintenance*, (1989)
28. *Tailings Dam Incidents*, (1994)
29. *Updated Guidelines for Selecting Seismic Parameters for Dam Projects*, (1999)

30. *White Paper on Dam Safety Risk Assessment: What Is It? Who's Using It and Why? Where Should We Be Going With It?*, (2003)

A.10 OTHER RESOURCES

Organizations

1. American Institute of Steel Construction (AISC), Chicago, IL.
2. American National Standards Institute (ANSI), New York, NY.
3. American Society for Testing and Materials (ASTM), W. Conshohocken, PA.
4. American Water Works Association (AWWA), Denver, CO. Website: www.awwa.org
5. Construction Specification Institute, Inc. (CSI), Washington, DC. Website: www.csinet.org
6. National Association of Corrosion Engineers (NACE), Houston, TX.
7. Concrete Reinforcing Steel Institute, (CRSI), Schaumburg, IL.
8. Society for Protective Coatings (formerly the Steel Structures Painting Council) (SSPC) Pittsburgh, PA.

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APPENDIX B
PROJECT INFORMATION

TABLE OF CONTENTS

B.1 INTRODUCTION

This appendix describes the various types and sources of information to be used during the planning, evaluation, and implementation phases for life extension and upgrade projects for dams and hydroelectric facilities. The assimilation of project information is an important step in assessing the viability of any planned projects of this type. Additionally, the assimilation of information could result in the identification of additional opportunities for improvement that could be achieved with low incremental effort or cost. While the committee attempted to make this appendix on project information as comprehensive as possible, it is recognized that obtaining all of the desired project information listed below is seldom practical or feasible. Individual project needs will ultimately determine what information is important and obtainable.

Project information can be viewed as two different categories:

- Background information - including the design basis and any upgrades to the project that have been incorporated.
- Current information – as a result of operation, maintenance, or inspection reports or studies conducted over the years.

Following the collection of the project information, a summary or condition assessment is performed that assimilates old and new information and determines the status of the structure.

Because service life extension and performance improvement projects are on hydroelectric facilities which have, presumably, performed adequately for many years, significant amounts of operational data may be available.

Project information may take many different forms. In the best case, records have been maintained including contract files, computations, drawings, geological reports, and other pertinent information. However, under typical circumstances, important records may have been lost, incorrectly filed in the archives, or disposed of during office moves or cleanups. A detailed search of the designer's and/or owner's historic records and files could prove helpful. Site visits to verify existing documents with field conditions will add to the project knowledge.

In summary, the various forms of project information that can be recovered will significantly assist in the planning, evaluation, and implementation phases for life extension and upgrade projects for dams and hydroelectric facilities. The return of the investment of time and funds on these efforts may be very high and allow project benefits to be expanded.

B.2 BACKGROUND INFORMATION

a) Original Design Basis Documents

The original design basis documents may provide the most complete and detailed compilation of information on engineering methods and assumptions used for the design of the facility. The availability of such documents will be dependent on the date of the initial design, on as-built construction records, and on changes made during the construction phase. On most projects, some very useful and revealing information can be obtained from photographs taken during construction.

The original design basis documents generally include geologic and geotechnical investigation reports, design criteria or standards, design computations, engineering drawings, engineering specifications, and procurement documents. The original design basis documents frequently allow determination of the design material properties such as shear strength of rock, friction angle and cohesion of soil, uplift assumptions for concrete sections, drain configuration and effectiveness, and the hydraulic design basis.

The original design basis documents may also provide information on the structural materials used, such as steel, concrete, and timber. Since the early 1900's engineering standard specifications have existed that define limits, chemical and/or material properties for these common construction materials. These historical standards provide a means to determine minimum original strength properties of structural steel and reinforcing steel. Other important information, such as weldability of structural steel, can be assessed from these original standards.

b) Geologic and Geotechnical Reports

From the beginning of modern dam construction, the importance of the geologic condition of the foundation has been recognized as a critical component of dam design. Therefore, it is likely that a written geologic report was prepared for the dam site. Additionally, geologic reports usually described quarries, which were the source of concrete aggregate or fill for embankment dams. These reports typically discuss the type of rock, excavation limits, strength properties, and seepage control methods. These reports also typically include recommendations on foundation grouting, cut-off trenches or keys, abutment slopes, and surface preparation.

Most of the earlier geotechnical investigations and testing programs would have been done in conjunction with the geologic investigation to locate a preliminary dam site. Additional investigations would have been undertaken to determine the final location, availability of construction materials (concrete aggregates, riprap, rockfill, and embankment materials), dimensions, and engineering properties (strength, permeability, and compressibility). Recommendations for seismic design may have been made, depending on the dam location. Investigations and testing would continue through construction, filling, and start-up. Depending on the age of the project, actual samples or photographs of samples may be available for viewing.

Although there are many approaches to the investigation of the construction materials and soil types, no single method provides all of the required information. Typical indirect methods include USGS maps, aerial photography, satellite imagery, and geophysical surveys either on the surface or down the drill hole. The direct method involves drilling holes and extracting samples for visual identification, laboratory analysis, or geophysical data correlation.

The specific geologic information available typically includes a description of the rock type, the geologic formations exposed on the site, the dip and strike of rock layers, the degree of fracturing and folding, extent of weathering, existence of filled or unfilled cavities, location of faults at or near the site, and the location of any weak seams. In the best situation, drilling logs were maintained that would allow review by an engineering geologist and allow the application of current knowledge for the assessment of a life extension and upgrade project.

For most major dam projects, engineering geologists oversaw the excavation, grouting, and foundation preparation of dams founded on rock. Typically, excavation drawings were prepared prior to construction and updated during construction to show the actual extent of excavation. Frequently, these drawings were included in the drawing files. Likewise, drilling and grouting drawings were prepared and updated. Daily excavation and grouting reports were also generally prepared; however, the probability that these documents were retained in the project files is much less likely.

Frequently, geologic conditions during construction resulted in some change to the outline drawings for the dam. These changes due to geologic conditions can sometimes be identified by examining the design drawings and revision logs on the drawings. For example, for dams founded on karstic limestone, limited areas of deep excavation were frequently necessary due to fissures and cavities. An unusual structural feature, such as a deep key section or a steel reinforced area may indicate an underlying feature that may have an effect on a rehabilitation project.

c) Hydraulic Design

Some life extension and upgrade projects could have an effect on the hydraulic performance of some of the project features such as the spillway, spillway apron, sluices, power intakes, tunnels, penstocks, turbines or canals. During the original design, hydraulic calculations were probably prepared and, for larger projects, physical modeling and testing of the feature may have been performed.

The hydraulic design for most projects was performed in the early phases of project development. For this reason, the hydraulic design was commonly not included in the detailed computation package for the structural features and equipment for a dam. The hydraulics work was also frequently done in water resource engineering groups and not in the dam design group. For this reason, locating these documents may require a different approach. In some cases, however, these reports have been maintained in agency and university libraries.

Physical modeling, if performed, was frequently done by an agency laboratory, or a university or private laboratory. For this reason, a formal detailed report of the testing was probably prepared. This report may still exist in the files of the testing agency or library at an agency or university. Locating these reports will provide valuable information, especially if minor existing hydraulically induced problems, such as cavitation or erosion, could be exacerbated by changes being made as part of a life extension and upgrade project. The report could also provide cost saving information of alternatives or alignments that were previously investigated but not implemented.

d) Hydrologic Reports

The hydrologic design basis for a project was one of the key elements of the conceptual design of a dam. These reports were routinely prepared at the feasibility level. They were a key component of the engineering and economic studies used in the decision to construct a dam. For this reason, a formal report probably existed. If the report continues to exist, it is likely that is not located with other detailed design documents, like the calculations and specifications. A good probability exists, however, that this report is included in an agency or university library.

The report will include information and sources for original hydrologic design. This would include rainfall data, precipitation and run-off assumptions, and hydrologic modeling techniques. For a life extension and upgrade project this data may provide insight into the original design basis and allow comparison to current hydrologic practices for probable maximum precipitation and probable maximum flood flow determinations.

e) Design Criteria

For life extension and upgrade projects, it is important to obtain as much relevant information as possible on the design criteria and methods used in the original design. For many of the older dams, the design of the dam components, especially the earth portions, was based on experience and preferences. However, the basic design criteria can sometimes be developed from review of existing reports and calculations. Additionally, the design practice at the time can sometimes be gleaned from articles and papers published in technical journals and periodicals from the period. Many major, and some small projects had very detailed reports published in proceedings of ASCE and other technical societies and a review of the indices for these proceedings is always worthwhile.

For concrete dams and spillway sections of embankment dams, the properties of the concrete and design assumptions for strength and allowable stresses can sometimes be obtained from existing calculations. Concrete has been in common use as a structural material since the early 1900's and the American Concrete Institute Codes were introduced as early as 1930.

The importance of hydrostatic uplift assumptions for evaluating the stability of concrete dam sections was first identified and introduced in the 1930's. This was closely followed by the introduction of drainage and grout curtains as a means to reduce uplift. The methods for evaluating the stability of concrete dams and the associated design parameters which were used, have varied. Current criteria are generally more conservative for some parameters such as the value of cohesion that may be used at both the concrete rock interface and at intermediate sections/joints in the concrete monolith.

The design parameters for structural steel have changed significantly over the years. As early as 1900, ASTM standards for steel were developed. Frequently, design drawings identified the grade of steel used for gates and appurtenant works. Information on the design parameters for various vintages of structural steel is available in the American Iron and Steel Institute Design Guide 15.

Concrete reinforcing steel has also varied since it was first introduced. The properties of the steel, including the cross-sectional shape and deformations have varied. In addition to changes in the bars themselves, the design requirements for splice lengths and confining reinforcement have changed significantly over the years making qualification of some structures more difficult. Information on properties of rebar in vintage structures is available from the Concrete Reinforcing Steel Institute.

The quantities, types, and locations of the construction materials may have dominated the entire design. However, the spillway selection was influenced by the magnitude of the floods to be passed. On streams determined to have a large flood potential, the cost of a large spillway was frequently a considerable portion of the total cost. On more recent dam projects, environmental laws and regulations will have influenced the design criteria for the type of dam, its dimensions, and location of the spillway and appurtenant facilities.

f) Design Calculations

As many as possible of the original design calculations, changes during construction, alterations after start-up, and repairs should be located in the owner's or design firm's files. Any life extension or upgrade project may require original design calculations to be re-analyzed using the most recent codes, specifications, and standards. Non-destructive or laboratory testing may need to be performed on representative samples if the structural adequacy or strength parameters are doubtful. When any pertinent design information is missing, as-built field measurements may need to be obtained in order to verify the adequacy of the structures in question.

If original design calculations can be located, it is most likely that they will be for the stability of concrete sections and structural calculations for piers, operating decks, and building. These types of calculations were commonly being prepared by civil engineers in the early 1900's. Additionally, an engineering stability drawing was frequently developed which documented the results of the analysis.

Stability computations and records for earth embankments, may not be as readily found since early work was frequently based primarily on experience and industry practice. Even when slip-circle analysis was started, the work was done on large paper using graphical techniques and in general these records are often difficult to locate.

Calculations on procured equipment are sometimes available in contract files, or design information can be obtained from drawing submittals. Procured equipment could be valves, gates, hoists, and numerous other mechanical/electrical items. Most agencies were conscientious about maintenance of contract records due to performance guarantees and potential claims. For this reason, there is some possibility that important information is in contract files.

g) Drawings

As many as possible of the original design, construction and as-built drawings should be located in the owner's or design firm's files. A check should be made at the project site to ensure that present day conditions exist as shown on the historical drawings as minor undocumented alterations may have occurred. As mentioned above, field verification should be obtained.

h) Design Guides and Construction Specifications

Many organizations, agencies, and engineering firms developed standard guides and specifications for use in their design and construction work. Many of these documents may have survived and can be obtained from libraries or archives of the organization, if they exist. In fact, in many cases, it can be difficult to interpret design computations since they were so closely tied to formulas and methods used in the guides and standards.

Design guides were frequently developed for stability analysis, earth pressures, concrete design, and embankment design. Since the 1940's additional standards and guides were frequently developed for embankment stability, seismic design, drainage filter design, blasting, grouting, flood prediction and routing.

i) Construction and Start-Up Records

Some daily construction records have little long-term value. These would commonly be production rates, labor costs, and delivery records. However, some construction records would be very valuable to a life extension and upgrade project. Unfortunately, many of the valuable records may have been disposed of along with the less valuable ones.

Some of the valuable records are photographs, concrete placement records, compaction records, and sieve analyses. Daily diaries for the construction engineer have sometimes survived and can provide useful information. In addition, the start-up or commissioning reports, if available may have value.

In summary, the original design basis documents are invaluable in the planning and evaluation of life extension and upgrade projects. Without these documents, the scope of work is commonly expanded to cover uncertainties in original design, or at a minimum, overly conservative assumptions have to be made.

j) Previous Rehabilitation, Modification, and Upgrade Projects

Many, if not most projects have had significant rehabilitation or modification work done in the more recent past. The records from these projects can provide an important link to the original design basis. They also provide detailed information on the design basis for the rehabilitation or modification.

The condition of the existing facilities is typically assessed and an audit performed for a new life extension or upgrade project. Past projects starting after initial operation began should be reviewed, and a time line developed showing past design changes, additions, and repairs.

k) Hydrologic Upgrades

Revised hydrologic studies were likely to have been performed in the 1980's or 1990's. Therefore, very detailed information on these studies and any modifications should be available. These studies likely involved the determination of an updated inflow design flood and a dam break analysis.

A large percentage of projects required modification due to these updated inflows. Additionally, increased downstream development frequently required reclassification of a dam with subsequent application of more conservative design requirements. For embankment dams, the upgrades required detailed investigations to determine soil properties for stability and seepage assessments. For concrete dams, non-destructive testing was frequently needed to justify design parameters. Therefore these upgrade project files provide a wealth of information for a life extension and upgrade project.

l) Seismic Upgrades

Seismic upgrades were also likely to have been done during the 1980's and 1990's. These upgrades are important for evaluation of life extension and upgrade projects because seismic upgrade work will almost definitely require the determination of in-situ materials properties, especially in-situ soil properties. The data obtained from seismic upgrade work are usually more than sufficient for the work on life extension and upgrade projects.

m) Power Waterways

Power waterways for the vast majority of projects have been upgraded or have had significant maintenance work performed on them. Intake gates, seals, butterfly valves, and many other components typically require significant work every 10-15 years. The maintenance and upgrade projects for these waterway components can provide significant information, or at least links to previous work that will allow identification of information for upgrade and life extension projects.

B.3 CURRENT PROJECT INFORMATION

Development of current project information can include the following topics. The goal is to review and assess the current condition of the facility to be upgraded, and if necessary, perform any new or special studies to support the life extension project.

a) Instrumentation Data Evaluations

Instrumentation data for dams should be reviewed as part of the evaluation of an upgrade and life extension project. The data is of importance for two primary reasons. First, it provides verification that the performance of the structure or feature is in substantial agreement with the design basis. Second, it assists in determining if planned changes or modifications will affect the performance of the structure. For example, a roadway for powerhouse access to be located near the toe of the dam, could affect the efficiency of the toe drain system.

b) Embankment Dams

An embankment dam may have had instruments installed to monitor and better understand its performance during and after construction. The type and number of instruments would depend on the size, complexity, and vintage of the structure. The main purpose of the instruments was to furnish reliable comparison of pore pressures and movements that actually develop with the values assumed by the designer. This data should be reduced and plotted for review, especially in those areas potentially affected by the new project. Instruments may also be used to provide data for future design or operational modifications.

Common types of instruments in use are as follows:

- Piezometers for measuring hydraulic pressures in both the embankment and foundation.
- Surface monuments to determine settlement and horizontal movement of the dam.
- Internal instruments for measuring stresses and strains, vertical and horizontal displacements in both the embankment and foundation.
- Seismic accelerometers to determine the reaction of the bedrock and embankment to seismic activity.

Seepage through embankment dams is to be expected but must be measured, monitored, and controlled. Records of flow and reservoir elevation should be plotted to ensure that the volume is not increasing inappropriately with the reservoir head. Additionally, review of instrumentation records should allow an assessment of the condition of a filter and drainage system if installed.

The data analysis should obviously be an ongoing aspect of the safety program for the dam. However, implementation of a new project provides the opportunity to correct any deficiencies in the dam's performance. Special care needs to be taken to assure that problems are not exacerbated.

c) Concrete Dams

Concrete dams can be monitored in much the same way, and for the same reasons, as embankment dams. Piezometers may be installed in the concrete and foundation to measure uplift pressures and confirm the functionality of grout curtains and internal drains. Surface monuments are installed to measure horizontal and vertical movements especially if the dam is constructed on a soil foundation. Depending on the age and condition of the dam, crack monitors may also be installed to check movements.

Many upgrade and life extension projects have an effect on the performance of concrete. For many dams, the powerhouse structure is part of the dam. Therefore, review of the instrumentation results is critical to ensure that the stability and safety of the structure is not adversely affected. For example, some hydropower upgrade projects require modifications to the draft tube. It is critical in these cases to evaluate the effect on uplift and relief drains on stability and safety during the unwatered condition.

d) Inspection Reports

Inspection reports for a dam or appurtenant work must be reviewed as part of an evaluation of an upgrade or life extension project. Every feature at a hydroelectric project has a finite life. The inspection reports are the most valuable source of information for determination of the condition and remaining life. Life extension activities must include all portions of the project, and the cost of future life extension activities on that feature must be considered. For example, it may not justify extending the life a hydropower unit for 50 years when inspections are showing wall thinning on the penstock.

With more development occurring around reservoirs and below dams, increased attention needs to be placed on the safety aspect of the operation, maintenance, and recreational use of dams, reservoirs, and their related facilities. Extensive surveillance and multiple inspections have become a fundamental element of the hydroelectric facility. Regularly scheduled visual inspections by operational personnel should be performed as often as monthly. Qualified engineers should conduct site inspections and review the plots for instrument readings as often as quarterly and perform special inspections after extreme events such as earthquakes and floods.

e) Embankment Inspections

Embankment inspections consist primarily of a detailed visual examination of the upstream and downstream faces, the downstream streambed and abutments. The reports for these inspections should provide the necessary information to assess the effects of an upgrade or life extension project on the embankment, or to identify opportunities to improve the performance of the embankment.

Many embankments have long-term problem areas where wet spots, abutment seepage, or boils form. These areas are normally closely monitored and may be instrumented. Increasing reservoir level or maintenance of higher reservoir level for longer periods may have a detrimental effect on the embankment. If these conditions are likely to occur, remediation should be included in the upgrade and life extension project.

f) Concrete Inspections

Concrete inspections consist primarily of a detailed visual examination of the concrete features at a project including non-overflow sections, spillway and aprons, powerhouses, training walls and a multitude of other features. The reports for these inspections should provide the necessary information to assess the condition and life expectancy of the concrete features in relation to the upgrade and life extension project being evaluated.

Many concrete features have long-term problems with weathering, cracking, and alignment. Some of these problems are primarily cosmetic and can be corrected or controlled through long-term maintenance activities. However, if some problems are rooted in foundation or concrete growth problems, long-term solutions may be limited and the project life may affect the feasibility of an upgrade or life extension project.

g) Gates and Water Control Structures

Inspections of gates and water control structures consist primarily of a detailed visual examination, operational testing, and corrosion assessments. Many of these inspections must be performed underwater by divers or remote operated vehicles. The reports for these inspections should provide the necessary information to determine if the condition and life expectancy of these features is compatible with the upgrade or life extension project. Also, the need for major maintenance as part of the project can be assessed.

Most water control features are steel structures. Some require periodic coating, which has frequently been performed less often than required. Additionally, many are continuously submerged and have been since project completion. These features have a high potential to affect the cost and feasibility of upgrade and life extension projects. Costs for repair/replacement of intake gate guides, sluice gate seals, intake gates, and operating machinery can be very high if there is a need for divers and cofferdams.

h) Power Waterway Inspections

Power waterway inspections consist primarily of a detailed visual examination, operational testing, and corrosion assessments of power waterway components such as penstock, tunnels, surge tanks, trash racks, butterfly valves, scroll cases, draft tubes, draft tube bulkheads, and draft tube drainage. The reports for these inspections should identify problems and opportunities relating to the upgrade or life extension project.

Power waterway features typically have a finite life of about 50 years. Therefore, all projects must use the inspection reports to assess their condition. Corrosion and wall thinning for the steel components must be considered. Additionally, some hydroelectric facilities do not have the control mechanism in the power waterway that current safety requirements demand. Frequently, on vintage plants, adequate provisions were not made for bulkheads and isolation valves to allow major maintenance or replacement of power waterway components. Repair or addition of these components should be considered for inclusion in upgrade or life extension projects.

i) Special Studies

For virtually every project, numerous studies have been performed over the life of the project. These studies may be related to the dam and onsite hydropower facilities, but may also be related to downstream development, flood risk, or environmental assessments. These studies likely provide a significant amount of information on the project features that may be useable for evaluation of upgrade and life extension projects.

j) Hydrologic Studies

The primary purpose of most dams is to create a basin to store and control streamflow. This stored water is then used for water supply, power generation, low flow regulation, flood retention, and irrigation. Records needed to manage the water resource will have been maintained for a long time period, will be up-to-date, and will have representative periods of low and high flows. In some instances, revisions may need to be made to existing records if the hydrologic character of the river basin has changed. This could have been caused by adding or removing flow from the basin, constructing additional dams upstream, or industrial or residential construction upstream that altered the land use.

k) Seismic Studies

In regions where seismic activity is an important design consideration, many studies may have been conducted and reference materials will be readily available. However, in areas of infrequent activity or little damage, design or analysis information has to be interpolated from seismic zone maps, which define the degree of probability and magnitude of an earthquake occurring at the location in question.

l) Environmental Studies

Dams, reservoirs, and their associated structures cause impacts on the environment. When conducting an environmental study, concerns could be grouped under headings such as natural environment, cultural environment, and the water resource. Many of the projects that have been relicensed had an Environmental Impact Study performed, which provides a good source of information. In many cases mitigation measures can be taken to offset any undesirable impacts. A life extension or upgrade could possibly be used to correct environmental deterioration.

m) Consultant Reports and Assessments

Since the 1940's, most larger hydropower projects have utilized owner's engineers or board of consultants to provide a high-level review of the design and construction activities for hydropower projects. These consultant reports, along with more recent reviews, may provide significant information for use in the evaluation of upgrade and life extension projects.

n) Independent and Hydroelectric Consultant Review

The dam and its appurtenant facilities must be maintained in safe operating condition throughout the life of the project. Surveillance through the years must also be conducted in such a manner that any physical change in the project facilities can be detected and promptly corrected. The owner's employees should conduct periodic surveillance inspections and reviews of instrument readings. However, independent consultants, qualified in hydroelectric design and operations, should be scheduled to conduct periodic and special inspections. Any corrective actions required should be handled in an expeditious manner.

o) FERC Part 12 Independent Consultant Reviews

Not only does the FERC require yearly inspections by its own staff on FERC regulated projects, the FERC also requires an inspection by an Independent Consultant at least every 5 years. Such inspections are to provide historical information and existing conditions on the project. Conditions to be reviewed and inspected include settlement, movement, erosion, seepage, leakage, cracking, internal conditions of stress, hydrostatic pressures, foundation drains, relief wells, and stability of critical sections of the structures and reservoir. After the inspection report is filed, the owner must submit a plan and schedule for resolving any proposed corrective measures.

p) Operation and Maintenance Procedures and Records

Operation and maintenance procedures can be used to identify problem areas and opportunities for improvement. Review of these records may allow identification of plant components that are not performing to standards or need to be replaced for long-term economic benefit.

q) Maintenance and Repair Records

By reviewing existing information concerning past additions, repairs and other significant events, a time line can be developed that may show repetitive problems. For example, maintenance records may show that a butterfly valve has been out of service many times during the life of the project and that consideration should be given to replacement of its operating machinery.

r) Computerized Maintenance Management Systems

Some projects or hydropower systems use computerized maintenance management systems for scheduling, tracking, verifying, and trending maintenance activities. Reports from these systems can be valuable in identifying problem areas and opportunities.

s) Interviews with Plant Personnel

Interviews with current and former employees provide a great potential for obtaining and finding important project information. Current employees in operations and engineering have significant knowledge of the project that may not be captured in documentation. Many of these employees may have moved to other positions in the company but still will be able to provide some historical insight into design basis and construction methods and operational history. Retirees are sometimes the best source of information especially, if they were involved in the construction activities.

t) State of Knowledge of Stakeholders

The evaluation of an upgrade and life extension project could include a study of the knowledge the stakeholders have in the project. Project authorization will involve technical, economic, and political assessments. The political assessments will, to a large part, depend on the knowledge that the stakeholders have of the project benefits, costs, and issues.

Owners may view a hydropower project much like a coal fired power plant and not recognize or appreciate the wide-ranging effects that the project has on the public. Water quality and reservoir levels typically drive the public's perception of a project, while owners are looking primarily at the electric power generation benefits. The engineer has to understand these perceptions in order to provide the appropriate presentation of the evaluation results.

u) Historic Performance of Dam and Hydroelectric Facility During Extreme Events

The performance of the dam and appurtenant works during extreme events provides valuable information for upgrade and life extension projects. Marginal performance during these events may allow the identification of opportunities to enhance safety or preclude costly repairs during future events.

Damage during historic seismic events should be assessed. Firstly, the damage and repairs should be assessed to determine if opportunities exist to update equipment. Secondly, the damage assessment will be used to ensure that the design basis for the new upgrade and life extension project adequately address the seismic history.

Damage from major flood events should also be assessed in a similar manner. Damage to aprons and stilling basins is common at some projects. The evaluation must consider the effects of the upgrade or life extension project on those features. For example, if the reservoir level is kept higher for longer periods of time after the upgrade, the duration of spillway use may be extended, resulting in increased damage.

APPENDIX C

TABLES

TABLE OF CONTENTS

TABLE C-1 TYPES AND USE OF GATES

TABLE OF CONTENTS, BY GATE TYPE [1]

FOOTNOTES FOR TABLE C-1

1) Gate Type: Gates are grouped into types that are similar in design or major mode of operation. Some gate types (ex. Radial, Vertical-lift Broome) are called by a multitude of names, and some gate types (ex: Vertical-lift) may include many different types of gates. The common names for most gates are listed. Some of the gates identified may be considered to be antiquated in that such gates have not be designed or manufactured since the 1960s.

2) Use: Not all gates of a type can be used for all of the purposes listed.

3) Spillway Discharge: Gate without the ability to provide a fine control or regulation of the discharge of a spillway or gate.

4) Spillway Control: Gate with the ability to be used to provide a fine control or regulation of the discharge of a spillway or gate.

5) Fail Safe: Some gates can be designed to be fail safe, defined as that position to which a gate will position itself (either automatically or under manual control) to control the release of flow in a manner that ensures the safe operation of the hydroelectric project. The fail safe position of a gate is dependent on its Use, with the fail safe position for gate use as follows:

   - Spillway gate (Discharge or Control): Open, to maintain flows and prevent an increase in the level of the impoundment.
   - Intake gate: Closed, to prevent an uncontrolled release of flow.
   - Emergency closure: Closed, to prevent an uncontrolled release of flow and loss of the impoundment, or to prevent damage or destruction of power generating or other mechanical equipment that may malfunction.
   - Sluice: Closed, to prevent an uncontrolled release of flow and loss of the impoundment.

a) Cylinder Gates	
Use [4] [5]: Intake Gate	
Advantages	**Disadvantages**
- Capable of controlling intake flows and large openings - Sometimes used to control flow to low head , open pit turbines - Simple low capacity hoist requirements	- Low head applications only - Possible vibration problems when discharging - Large gates require counterbalance to reduce hoisting forces

b) Flashboards	
Use [3]: Spillway Discharge See Section 6.4 for additional information	
Advantages	**Disadvantages**
- Inexpensive means to provide spillway discharge capacity - Low technology - Unlimited length and geometry - Can be designed for automatic and/or manual release - Can be designed in hinged panels to improve release and resetting - Can be constructed of wood or metal	- Can be installed to not fail, resulting in increased head ponds and hydrostatic loads on water retaining structures - Worker safety during replacement or maintenance - Must be manually reset, only under low or no discharge - High operating and maintenance cost - Difficult to design and maintain to automatically fail at a specified water level - Discharge generally uncontrolled - Susceptible to loss by impact from ice or debris - Susceptible to partial loss or displacement under winter conditions with pond fluctuation - Tends to have high leakage (in freezing climates results in ice buildup an potential inoperability) - Frequent replacement of failed or deteriorated material

c) Fuse Gates	
Use (3) (5): **Spillway Discharge**	
Advantages	**Disadvantages**
- Automatic (fail safe) operation, no monitoring required - Simple construction - Low maintenance - Operates as fixed weir until it tilts - Designed to tilt at specific water level - Due to labyrinth layout, under low heads it can increase the length and discharge capacity of the spillway - Economical means for providing increased discharge capacity for large floods - Simple means to increase storage capacity of a dam - Economical solution for use as small side channel dam - Applicable for use in unsophisticated environment	- Not suitable for pond level control - Not suitable for floods with low return frequency - Not suitable for discharge of floating materials - Gates have to be removed from riverbed after tilting - After tilting, gates have to be reset onto dam to restore pond level - Recovered gates may not be reusable

d) Hinged Crest Gates (self operating)	
Use (4) (5): **Spillway Control** (Drum & Sector gates) **Water Level Control** (Bear-trap Gates) **Also known as**: - Bear-trap Gates (a.k.a. Roof-Weir) - Drum Gates (hinged on upstream side) - Sector Gates (hinged on downstream side) - Stickney Gates (drum gate)	
Advantages	**Disadvantages**
- Self operating via hydrostatic pressure, providing fail safe operation - No outside source of power required for operation - Overflow discharge - Provides unobstructed flow - Suitable for discharging floating debris	- Complex gate - Height limitations - Require extensive / complex civil works - Not suitable for use on low dams - Seal design and maintenance is critical - Not suitable for use if gate is submerged by tailwater - Control system critical - Susceptible to siltation of gate recesses and controls - Enclosed downstream face results in confined space entry requirements - Limited use (antiquated design)

e) Hinged Crest Gates (motorized)	
Use [4] [5]: **Also known as:**	**Spillway Control** - Bascule Gates - Bottom Hinged Gates - Flap Gates • Fish-belly • Pelican
Advantages	**Disadvantages**
- Can be designed to open under gravity without power, providing fail safe operation - Overflow discharge - Suitable for discharging floating debris - Certain types can be installed in sections of unlimited number/length - No visually intrusive overhead structure	- Height limitations - Requires accurate side sealing surfaces - Difficult seal access and maintenance - Hinge bearings not easily accessible - Environmental concern with use of hydraulic oils - Limited access to operating cylinders if installed underneath the gate - Bearings design for immersed or submerged conditions - Difficult to isolate one length of gate if there are no end (closure) piers between gate sections - Enclosed downstream face results in confined space entry requirements

f) Maintenance Gates	
Use [5]: **Also known as:** **Types of Gates:**	**Temporary** - Bulkhead Gates - Guard Gates - Stop Gates - Slide - Wheel/Roller (includes: Stoney, Broome, Caterpillar Tractor) - Stoplogs (steel, wood)
Advantages	**Disadvantages**
- Low construction cost	- Not suitable for flow control - Requires external/mobile lifting/transportation device - Storage and rigging issues

g) Needle Gates	
Use [(3)]**:** **Spillway Discharge** **Also known as:** - Needle Beams - Stanchion Stoplogs	
Advantages	**Disadvantages**
- Simple, low technology - Low construction costs - Manually operated, power not necessary for release	- Not suitable for flow control - Not suitable for pond level control - When in place, not suitable to discharge floating debris - Release results in loss of depth of retained pond - Must be manually removed / released to discharge - Must be manually reset, only under low or no discharge - Limited heights - Requires personnel access deck along the top

h) Radical Gates (motorized)	
Use [(4)]**:** **Spillway Control** **Intake Gate** **Sluice Gate** **Also known as:** - Taintor (tainter) Gates - Sector Gates	
Advantages	**Disadvantages**
- Low susceptibility to jamming by debris - Few moving parts - No unbalanced forces - Absence of gate slots - Low hoisting force - Mechanically simple - Simple seal design - Bearings out of the water - Can be fitted with overflow section - Some inspection possible with gate in service - Gate lengths up to 150 feet - Gate heights up to 65 feet, depending on width - Lower construction and installation	- Requires power to open, therefore not fail safe - Trunnion bearing design, friction - Difficulties in inspecting & maintaining trunnions - Maintenance of lifting cable / chain / rollers - Limited ability to discharge floating debris - Extended flume walls / piers - Increased fabrication complexity - High concentrated loads - Installation difficulties - Winter operation (freezing) - Enclosed downstream face results in confined space entry requirements

costs than for other types of gates of comparable size - Gates can be equipped with a top seal for use as (submerged) intake control gates	- Ice build accumulation on lifting chains - Vibration when discharging - As an intake gate; gate size, gate chamber requirements, inaccessibility, maintenance

i) Radical Gates (automatic/float operated)	
Use [4] [5]:	**Water Level Control** **Spillway Control**
Advantages	**Disadvantages**
- No outside source of power required, providing fail safe operation - Absence of machinery	- Height limitations - Wide piers to accommodate displacers, counterbalances, or floats - Can malfunction due to blockage of inlet or control system - Counterbalance visually intrusive - Winter operation (freezing) - Generally smaller that motorized radial gates

j) Rolling Weir Gates	
Use [4]:	**Spillway Control** (vertically lifted gate)
Advantages [5]	**Disadvantages**
- May be designed as overflow or underflow gate - Can be manufactured to very wide widths	- Complex to fabricate - Effectiveness and longevity of side seals - Requires accurate side sealing surfaces - Debris lodging in racks - Limited (antiquated) use

k) Rubber Dams	
Use [4] [5]:	**Spillway Discharge** Also includes pneumatically operated panels. See Section 6.4 (pneumatic flashboards) for additional information
Advantages	**Disadvantages**
- Unlimited length - Can provide fail safe operation - Overflow discharge - Suitable for discharging floating debris	- Height limitations - Limited flow control - Difficult to take a length or section out of service

- Simple mechanical system with low power demands - Manual or automated control - Simple civil works	- Limited life expectancy - Potential vandal damage - (Possible) ice and debris issues

I) Vertical-Lift Gates	
Use [(2) (3) (4) (5)]	**Includes Gates Known As**
- Spillway Control Gate - Intake Gate - Outlet Gate - Sluice (water) - Trash Sluice Gate - Sediment Sluice Gate - Emergency Closure Gate - Maintenance Gate - Guard Gate	- Bonnet - Broome/Caterpillar/Coaster/Tractor - Free-flow - Jet-flow - Leaf or Hook (multi-section) - Ring-follower/Ring Seal/Paradox - Rolling Weir (discussed as a specific type of gate) - Slide - Stoney - Wheel/Roller
Advantages	**Disadvantages**
- May be raised or lowered to open - Simple fabrication for certain gate types - Suitable for high heads - Short piers - Numerous types of gate operators - Can be fitted with overflow sections - For normally open operations, can be designed to close under gravity without power (fail safe operation) - Can be designed to open under high, unbalanced heads	- Importance of seal design - Bearing maintenance - Winter operation (freezing) requires heating of gate slots - Gate slots required (problems at high velocity) - Susceptible to jamming by debris – both opening and closing (especially shallow gates) - High hoisting load unless counterbalanced - Overhead support structure visually intrusive - Needs air admission for discharges into a passage - Cavitation under high head - High discharge velocities - Ring-follower: Large overall height, cost, requires regular flushing and drainage of bonnet - Broome and Ring-follower gates have numerous moving, submerged parts - May require wheels on the sides of the gate to prevent lateral movement (especially for high head wheel gates)

m) Wicket Gate	
Use [(3)]**:** Sluice Gate Intake Gate **Also known as:** Valve Gates. Excludes navigation wicket-type gates.	
Advantages	**Disadvantages**
- Simple, vertically aligned disc with multiple discs used for wide openings	- Obstructs passage of debris - Operation susceptible to jamming and obstruction by debris - Difficult to clear obstructed gate - Low head applications only - Limited (antiquated) use

TABLE C-2 VALVE TYPES AND APPLICATIONS

TABLE OF CONTENTS, BY VALVE TYPE [(1)]

FOOTNOTES FOR TABLE C-2

1) Valve Type: Valves are grouped into types that are similar in design or discharge function. Some valves (ex. Fixed-cone) are called a multitude of names, and common names are listed. Some of the valves identified may be considered antiquated in that the valves have not be designed or manufactured since before 1950.
2) Use:
   - Isolation Valve:
     Valve used typically as a guard for emergency closure or maintenance purpose. Intended for operation in either the open or closed position, not suitable for flow control.
   - Flow Control Valve:
     Valve that can be used to control flow. Usually has an isolation valve as backup.
   - Discharge Control Valve:
     Energy dissipating valve, controlling terminal discharge although some can be used in-line. Energy dissipation by air friction and entrainment. Usually has an isolation valve as backup.

3) Operating Heads:
- Low: less than 150 feet
- Medium: 150 to 1,000 feet
- High: greater than 1,000 feet

a) Butterfly Valve	
Use (2) (3): Isolation Valve	
Advantages	**Disadvantages**
- Most common isolation valve - Simple, compact design - Easy to maintain - Relatively low loss coefficient with solid disk (stub shaft) geometry - Lower loss coefficient with flow-through disk (non-stub shaft) geometry - Closure by gravity can be arranged with counter weights - Available in large sizes - Medium operating head - Can be supplied with a secondary (inflatable) maintenance seal	- Normally opened under balanced heads - Possibility of blade flutter - Possibility of eddy shedding from blade tips or from pivot shaft - Valves designed per AWWA standards have higher head losses - Flow velocities limited to 15 to 20 feet per second

b) Clamshell Gate	
Use (2) (3): Discharge Control Valve	
Advantages	**Disadvantages**
- Installed at end of pressure conduit - Discharges under free or submerged conditions - Operates under wide range of flows without vibration - Simple operation - Multitude of operating mechanisms (hydraulic is most convenient) - High discharge coefficient - Medium operating head - Low operating forces	- Requires isolation gate or valve - Non-energy dissipating - Requires stilling basin/plunge pool for energy dissipation - Not suitable for in-line service unless an expansion chamber is included downstream on the valve - Operating mechanism must maintain pressure on gate to minimize leakage - Ice accumulation due to seal leakage - Difficult to design and fabricate

c) Fixed Cone Valve

Use (2) (3): **Discharge Control Valve**
Also known as: - Howell-Bunger Valve
- Hollow Cone Valve

Advantages	Disadvantages
- Very efficient energy dissipater - Stilling basin not required (many installations utilize stilling basins to dissipate the energy remaining in the valve's discharge) - Simple construction - Relatively low cost - Medium operating head - Can be operated electro-mechanically or by hydraulics - Good discharge coefficient - Available in large sizes - Least flow obstruction of discharge type valves therefore not as susceptible to debris trapping as other discharge valves - Excellent aerator	- Seal of sliding sleeve may leak - Vibrations - Vane failure - Requires adequate air supply during operation (air is entrained into the discharge spray for energy dissipation) - Splash back may be concern in freezing climates, and may necessitate the use of electric heaters to allow the valve components to move freely - Requires long straight approach - Spray from discharge jet - Often has a discharge hood downstream of the valve to confine / contain the discharge; the hoods (concrete and or steel lined) subject to erosion and failure due to high discharge velocities - Noise of discharge - Not suitable for submerged discharge

d) Hollow Jet Valve

Use (2) (3): **Discharge Control Valve**

Advantages	Disadvantages
- Can be installed directly after bend in penstock - Medium operating head	- Higher coefficient of discharge than fixed cone valves, therefore a less efficient energy dissipater - Discharges into stilling basin - Greater cost than fixed cone valve - Fluid passages can become blocked by debris - Internal moving parts - Operates on internal pressure differences - Difficult inspection access - Servicing requires removal of valve

e) Needle Valves	
Use (2) (3):	**Flow Control Valve** **Discharge Control Valve**
Also known as:	- Doble - Ensign - Balanced - Interior differential - Lanier-Johnson
Advantages	**Disadvantages**
- Can be used as an in-line pressure reducing valve - Can be operated without external power - Medium to high head operation - Large sizes, up to 13 feet - Positive closure/seating - Closes droptight	- Higher coefficient of discharge than fixed cone valves therefore as a discharge valve a less efficient energy dissipater - Complex operation and maintenance - Qualified maintenance staff required for complex operation - Low coefficient of discharge - Greater cost than hollow cone valve - Fluid passages can become blocked by debris - Internal moving parts - Operates on internal pressure differences - Susceptible to slamming closures if improperly operated - Sluggish or incomplete closing at low reservoir levels - Inspection and servicing requires removal of valve

f) Sleeve Valves	
Use (2) (3):	**Flow Control Valve** **Discharge Control Valve**
Valve Types:	- Perforated cylinder type - Multi-jet - Multi-port - Sliding Plate
Advantages	**Disadvantages**
- Medium to extreme high head operation - Medium diameters (<10 feet) - May be used as bypass valve - May be installed in-line or as a terminal discharge valve - Excellent flow control or energy dissipater - Submerged or free discharge	- Orifices in perforated cylinder can be blocked by debris - Internal moving parts - Inspection and servicing sometimes requires removal of valve

g) Sluice Valves	
Use [2] [3]:	**Isolation Valve**
Also known as:	- Gate Valve - Knife Gate - Jet Flow Gate

Advantages	**Disadvantages**
- Common valve - Best suited for open or closed operation - Low cost - Simple and reliable - Low to low-medium head operation for sizes <13 feet - High to very high head operation for sizes <3 feet - Unobstructed fluid passage when fully open, very low loss coefficient	- Valve blade unsupported during raising and lowering - Eddy shedding from blade tip - Only suitable for open & close operations under low velocity flows - High headlosses when in partial open positions

h) Spherical Valves	
Use [2] [3]:	**Isolation Valve**
Valve Types:	- Rotary - Ball - Cone - Plug

Advantages	**Disadvantages**
- Simple design - Easy to maintain - Unobstructed fluid passage when fully open, very low loss coefficient - Closes droptight - Closure by gravity can be arranged with counter weights - Manufactured in large sizes - High head operation - Very high head operation for small sizes (< 6 feet) - Can be supplied with a maintenance seal - Fixed or movable seals	- Higher cost than butterfly valves - Requires external power source for rotational closure - Movable seals are a problem

TABLE C-3 CAUSES OF COMMON VALVE PROBLEMS

DESIGN AND MANUFACTURING	MAINTENANCE	OPERATION
- Inappropriate application for a particular valve - Over stressed parts or subassemblies - Fatigue - Excess deflections - Design prohibits access for maintenance or routine inspections and testing - Design promotes rusting - Design promotes galvanic corrosion - Design prohibits replacement of worn parts - Poor design geometry or design features - Inferior materials, poor castings, cracks, plate laminations, voids or inclusions - Poor welding (lack or insufficient post weld stress relieving) - Poor machining - Inappropriate or sub-standard coating systems or valves use - Poor coating application - Inappropriate valve operators and/or controls	- Lack of routine inspection - Lack of routine testing - Lack of proper maintenance, cleaning, adjusting, or lubrication - Lack of aggressive coating program - Improperly engineered repairs - Poor welding (lack or insufficient post weld stress relieving) - Inappropriate valve operators and/or controls - Inappropriate modifications to valve operators or controls - Failure to inspect following emergency or severe operation events	- Exceeding operational design limits - Lack of proper application knowledge - Improper operation - Lack of routine and periodic exercising - Lack of training - Poor operational procedures

TABLE C-4 FLASHBOARD SYSTEMS

a) Simple

Advantages	Disadvantages
- Low initial cost - Simple and commonly available materials	- Must be manually reset - Impoundment must be lowered to replace boards - Potential for excessive lost generation until boards are reset - Moderate labor expense to replace boards - Requires safe access to spillway / flashboards by operations personnel - Moderate material costs to replace boards especially if flashboards fail frequently - Tripping of boards requires failure of a structural element (i.e. bending of vertical steel pin) which can result in poor control as to when the boards will fail - Release of large quantities of flashboard and flashboard support material downstream may be environmentally unacceptable and costly to remove, if required - Highly susceptible to debris and ice, prematurely tripping the boards - Wood flashboards must be sealed to minimize leakage - Leakage under freezing conditions may prevent failure - May be susceptible to being lifted if frozen impoundment fluctuates during winter

b) Manually Tripped

Advantages	Disadvantages
- Low initial cost - Can be tripped at any impoundment elevation, providing a limited control of when discharge flow is increased - Can also be designed (within limits) to fail	- Disadvantages of the simple flashboard system are applicable - System often not designed to fail on its own - Potential threat to the integrity

at a pre-determined pond level - May be panelized and hinged - Low material costs to reset boards if boards are panelized and hinged - Lower labor costs to reset boards if boards are panelized and hinged	and stability of the water retaining structures if the flashboards are not released when intended

c) Shear Pin Trippable System	
Advantages	**Disadvantages**
- Can be designed (within limits) to fail at a pre-determined pond level - Low to moderate initial cost - Low labor expense to reset boards - May be panelized and hinged - Low material costs to reset boards	- Requires failure or release of a structural element (i.e. a shear pin in this example) - Tends to fail, due to debris impact, before the pre-determined pond level - Continually rebuilding may result in unanticipated design modifications in the field preventing the system from failing when intended, threatening the integrity and stability of the water retaining structures - Must be manually reset - Impoundment must be lowered to reset boards - Potential for excessive lost generation until boards are reset

d) Hydraulically Operated	
Hydraulically operated flashboard systems are similar in operation and function to Crest Gates.	
Advantages	**Disadvantages**
- Can be manually or automatically raised and lowered at any time regardless of impoundment elevation - Can be lowered (automatically or manually) without need of power source (fail safe) - Can be installed in long lengths - No labor cost to reset boards - No material expense to reset boards - Not susceptible to premature failure by	- High initial cost of system - Significant cost of civil works to retrofit an existing dam - Close proximity of oil products to impoundments and rivers - Hydraulic lines susceptible to damage by debris impact when discharging - Difficult to isolate a section of flashboard to facilitate repairs

d) Hydraulically Operated	
Hydraulically operated flashboard systems are similar in operation and function to Crest Gates.	
Advantages	**Disadvantages**
debris or ice - Can mitigate environmental problems in and around the impoundment caused by excessive pond fluctuations associated with simple flashboard systems - Can be designed to (open) discharge in definable lengths - Overflow type discharge can be used to sluice debris and ice	- Requires maintenance of bottom hinge seals and seals between adjacent flashboard panels - Leakage under freezing conditions may prevent / inhibit operation

e) Pneumatically Operated	
Includes: Inflatable "Rubber dams" (i.e. Obermeyer Gates and Bridgestone inflatable dam)	
Advantages	**Disadvantages**
- Low cost of civil works to retrofit existing dam - Can be installed in long lengths - Can be manually or automatically raised and lowered at any time regardless of impoundment elevation - Can be lowered (automatically or manually) without need of power source (fail safe) - No labor cost to reset boards - No material expense to reset boards - Not susceptible to failure by debris or ice - Can mitigate environmental problems in and around the impoundment caused by excessive pond fluctuations associated with simple flashboard systems - Certain manufacturers' pneumatic flashboards can be designed to (open) discharge in definable lengths - Certain manufacturers pneumatic flashboards can be used for sluicing debris and ice	- Moderate to high initial cost - Difficult to isolate a section of flashboard to facilitate repairs - Intermediate piers may be required - Possible limitation for installation on curved spillways

TABLE C-5 FLOOD-PROOFING MEASURES AND CONSIDERATIONS

a) Powerhouse Protection under Normal Flooding Conditions
Under Normal Flooding Conditions, the powerhouse can be protected by:
Standby electrical backup power (See subsection on Electrical Backup Power)
Duplex pump installations
Interconnection (motorized, fluid-power, or manual valve) of drainage to dewatering sump
Dewatering sump shaft extending to level above maximum design tailwater
Dewatering sump shaft extending to cover slab with leak-tight, bolted manhole cover and vented to level above maximum design tailwater
Maintenance and testing of high-water, sump-surface alarms and float switches for pump-on and pump-off automatic operation
Maintenance of check valves
Check valves in discharge lines from dewatering sump to draft tubes, penstocks, tunnels, shafts, tailwater, headpond, and other high-potential receiving zones
Requiring manual, fluid-power, or motor-operated valves in all lines from dewatering sump to higher-potential discharge points
Having at least two separate sumps, one drainage and one dewatering
Maintaining screens between water passages and valves on lines to dewatering sump to prevent line and valve-disc fouling
Provision for pneumatic blow-off of debris collecting in lines, valves and on screens in dewatering lines
Cleanout fittings
Isolation valves for maintenance of main valves on lines to and from dewatering and drainage sumps
Heat tracing on lines and valves that might freeze
Bulkhead separation of auxiliary galleries from powerhouse interior passages
Separate (independent) drainage sumps in auxiliary galleries

b) Powerhouse Protection Under Extreme Flooding Conditions
Under Extreme Flooding Conditions, the powerhouse can be protected by:
Moveable, fabricated bulkheads and permanent, retrofitted frames and supporting structures (primarily perimeter doors, windows, and hatches)
Modular freeboard (sand bags, deck flashboards, stone-filled concrete masonry units, stoplogs and needle posts)
Moveable, fabricated, isolation bulkheads and permanent, retrofitted frames and supporting structures (primarily interior doors, wall openings, and hatches)
Backup submersible pumps with discharge hose and couplings
Portable pumps put in station sumps to augment duplex standard with appropriate discharge hose, valves and fittings
Oversized pumps, discharge line, valves and fittings instead of duplex standard in sumps
Tie-downs (anchors and ballast) for lubrication and fluid power tank reservoirs (usually at turbine pit level or below)
Oil reservoir vents fitted with float valves
Isolation bulkheads and pneumatic plugs for turbine pit areas
Relocation of electrical and electronic equipment from sub-tailwater level to higher elevations
Entrance towers or penthouses, and stack vents, for elevating air-intake and exhaust louvers and flues (common among powerhouses designed for overtopping)
Liquid-immersion MST (main step-up transformer) within powerhouse substructure with HVAC, oil containment, and fire suppression systems in isolation vault
Bolt-down or ballasted and sealed hatch covers over equipment maintenance shafts (common among powerhouses designed for overtopping)

c) Project Flood-Proofing Features and Checklist

PROJECT FEATURE	COMPONENT	CHECK FOR:
Intake	Water passage vents Instrument air vents (breathers) Trolley rails and wheels Power, control and instrumentation, conduits & outlets Exposed bus bars Gate hoists	Bypass of closed gates & valves False readings & fouling Overtopping debris fouling Defective seals & fittings Phase-to-phase fault Emergency power

c) Project Flood-Proofing Features and Checklist

PROJECT FEATURE	COMPONENT	CHECK FOR:
Roof	Equipment hatches Power and control conduits Stairwell hatches Air intake & exhaust vents Penthouse doors and windows Parapet wall scuppers	Seal & gasket leakage Defective seals & fittings Seal & gasket leakage Low elevation & bulkheads Bulkheads and shutters Clogging & ponding
Superstructure Perimeter	Pedestrian and service bay doors Windows Conduit penetrations Ventilation louvers Utility penetrations Air changes Veneer weeps	Bulkheads Bulkhead shutters Seal & gasket leakage Damper motor failure Seal & gasket leakage Ventilation fan failure Wall cavity flooding
Substructure Perimeter	Sump discharge outlets Draft tube gate seals Draft tube gate condition Draft tube gate operators Gallery passage interconnections Underdrain interconnections	Check valve condition Seal defects & loss Loss of mass, corrosion Rated torque, bearings, gears No bulkheads Unknown sump inflows
Interior Galleries	Isolation doors Floor hatches Pipe and conduit penetrations Interconnected drainage Ventilation penetrations Air change Gutters & floor drains	Bulkheads Bulkheads Seals & gaskets Unknown connections Duct short-circuits[1] Ventilation fans Drainage short-circuits[1]
Spillway	Gate operating deck Traveling hoists Fixed hoists Pier operator chases Exposed bus bars Lighting Lightening protection	Overtopping fouling Rail & trolley wheel fouling Overtopping fouling Operator gallery short circuiting[1] Phase-to-phase faults Reliability in emergencies Grounding
Galleries	Air and water lines Lighting bus Gutters, observation wells, gravity drains, and exterior access passages Drainage sumps	Access, floatation Ground fault Short circuits[1] Pump failure, outlet short circuit[1]

(1) The term short-circuit refers to any unexpected route or pathway of flooding. An expected pathway, such as a service-bay doorsill at grade, might be protected with a bulkhead retrofit, but water wells up from within the powerhouse from unexpected sources. The pathway may be long and circuitous, but in terms of potential, water is taking the shortest pathway available; thus, the phrase short circuit.

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APPENDIX D

SUMMARY OF CASE HISTORIES

TABLE OF CONTENTS

TABLE D–1: INTAKES – SECTION 4.2.4

TITLE – PROJECT - REFERENCE	PROBLEMS	SOLUTIONS
1. High Cost of Intake Operation and Maintenance Naches Project (ASCE, 1995)	- Intake gates do not seal properly - Gate operators were to low to use during high flows - Intake had no trash racks - Need for frequent cleaning of rock trap - Debris lodged in and against gate slots - Unacceptable head losses - Frazil and sheet ice	- New gates installed - Gate operators were located to high level - Tunnel added to provide additional flow capacity, thus reducing velocities and head losses - Installed trash racks - Added a trash rake - Lights installed to facilitate night operations
2. Increased Hydraulic Capacity Mill "C" Project (ASCE, 1995)	- Increase flow capacity of intake without creating poor hydraulic conditions	- Construct a new intake
3. Hydraulic Approach Modifications Upper Mechanicville Project (ASCE, 1995)	- Impact of flow conditions on navigation and turbine performance	- Modified guide wall to address navigation - Constructed partial depth guide vanes to address performance
4. Control of Water Temperature Shasta Dam (USBR, 1997)	- Inability to provide cool water to salmon fishery	- Installed a multi-level intake structure
5. Seismic Upgrade of Low Level Outlet Elsie Dam (BC Hydro, 2004)	- Seismic induced failure of intake tower - Seismic induced failure of conduit and possible failure of Saddle Dam - Loss of ability to reservoir drawdown	- Replaced intake tower - Relined conduit - Added berm to toe of Saddle Dam - Refurbished discharge valve and added an isolation valve upstream of discharge valve

TABLE D–2: CONCRETE DAMS AND SPILLWAYS – SECTION 4.3.6.A

TITLE – PROJECT - REFERENCE	PROBLEMS	SOLUTIONS
1. Spillway Surface Deterioration Inghams Hydroelectric Development (Brookfield Power New York, 1997)	- Cracked concrete - Deep gouges in spillway surface - Spalling concrete - Wood flashboards that were unreliable, dangerous, and expensive to replace - Significant upstream pond fluctuations and resident complaints	- Resurfaced spillway with concrete - Installed inflatable dam
2. Dam Instability Due to Flood Loading Niagara Hydroelectric Project (AEP, 1997)	- Inadequate factors of safety	- Increased dam's mass with roller-compacted concrete
3. Dam Instability Due to Ice Loading Twin Branch Hydroelectric Project (AEP, 1999)	- Dam instability due to ice loading	- Installed an air bubbler on the upstream side of the dam
4. Insufficient Spillway Capacity Brule Dam (MWH, 1988)	- Insufficient spillway capacity	- Rehabilitated existing tainter gate controlled spillway - Resurfaced spillway concrete - Constructed new two stage fuse plug controlled, side channel auxiliary spillway

TITLE – PROJECT - REFERENCE	PROBLEMS	SOLUTIONS
5. Inadequate Seismic Stability & Spillway Capacity Devil's Gate Dam (MWH, 1997)	- Inadequate stability - Inadequate discharge capacity	- Replaced existing ungated spillway with an orifice and overflow spillway headworks - Added a concrete spillway chute - Soil-cemented and anchored foundations - Added roller compacted concrete buttresses - Added foundation drains - Added a seepage cutoff wall
6. Erosion of Unlined Discharge Channel L.L. Anderson Dam (MWH, 1999)	- Severe erosion of unlined discharge channel	- Lowered and widen spillway channel - Creation of hydraulic drops to dissipate energy - Added a plunge pool and weir - Added channel walls to direct flows
7. Concrete Deterioration (Alkali-Aggregate Reactivity) Lake Decatur Dam (MWH, 2001)	- Deterioration of concrete due to potential alkali-aggregate reactivity	- Added expansion joints - Epoxy grouted cracks
8. Instability (High Uplift Pressures) Harlan County Dam (USACE, 1975)	- Dam sliding on bentonite seam	- Installed drain holes in existing gallery

TITLE – PROJECT - REFERENCE	PROBLEMS	SOLUTIONS
9. Insufficient Spillway Capacity and Stability Wahleach Hydroelectric Project (Hartford, 1998)	- Insufficient spillway capacity - Structurally unstable spillway	- Overflow ogee weir was replaced - Spillway chute was added - Sheet-pile cut-off walls was added - Early Notification (warning) System was installed
10. Concrete Deterioration and Insufficient Dam Stability Rainbow Dam (Kleinschmidt, 1992)	- Freeze-thaw deterioration - Concrete erosion - Insufficient factors of safety	- Routing of leakage - Mass concrete added to address stability and deteriorated concrete
11. Soluble Soils & Embankment Dam Failure Quail Creek Dam (UT DWR, 2002)	- Leakage - Soluble gypsum foundations - Dam failure	- Reconstruction using roller-compacted concrete
12. Alkali Aggregate Reactivity Fontana Dam (Meisenheimer & Wagner, 1997)	- Heavy cracking - Alkali Aggregate Reactivity	- Install post-tensioned anchors - Install stress relief slots
13. Alkali Aggregate Reactivity Hiwassee Dam (Meisenheimer & Wagner, 1997)	- Alkali Aggregate Reactivity - Cracks - Concrete spalling - Binding of radial gates	- Tools developed to predicate future AAR growth - Install post-tensioning anchors - Installed stress relief slots - Installed permanent cofferdam for future slot cutting

TITLE – PROJECT - REFERENCE	PROBLEMS	SOLUTIONS
14. Foundation Leakage 'El Cajon', Francisco Morazan Project, Honduras (Manguarian, 1994)	- Cracks in cement grout curtain - Seepage through and erosion of clay pockets in limestone foundation - Karst and fissures in foundation rock	- Injected wood, and concrete filled plastic and balls into grout holes - Inserted polypropylene feed sacks into grout holes
15. Insufficient Spillway Capacity and Seismic Instability, Buttress Dam Big Dalton Dam (MWH, 1999)	- Insufficient spillway capacity - Seismic instability of arched buttress dam	- Added morning glory type intakes - Infilled arches and buttresses with concrete
16. Flooding of Drainage Gallery and Insufficient Stability Leesville Dam (Kleinschmidt, 1998)	- Flooding of drainage gallery due to high headpond and tailwater conditions - Changes in criteria for uplift pressures - Inadequate stability due to uplift pressures - Inadequate stability due to increased hydrostatic loading	- Site specific PMF study was performed to upgrade flood discharge and stage requirements - Parapet wall was removed to lower hydrostatic flood loads - Post-tensioned anchors were installed - Watertight hatches and bulkhead doors were installed - Installed flood proof back up power generator, dewatering pumps, and control systems - Developed plan for abandonment under flood conditions

TABLE D–3: EMBANKMENT DAMS – SECTION 4.3.6.B

TITLE – PROJECT - REFERENCE	PROBLEMS	SOLUTIONS
1. Foundation Liquefaction Lopez Dam (MWH, 2002)	- Foundation liquefaction - Unsafe embankment dam under seismic loading	- Stone columns - Add downstream buttress and widen crest
2. Toe Erosion at Outlet Works Milford Dam (Dridge & Matthews, 1997)	- Erosion of channel banks - Piping of the dam foundation	- Deepening outlet channel - Restoring riprap on channel bans - Added rock toe with multiple bedding layers to filter seepage - Added a channel stabilizer
3. Insufficient Spillway Capacity and Seepage He Dog Dam (MWH, 1995)	- Undersized spillway - Seepage though abutment - Artesian pressures - Inoperable outlet works	- Added new roller-compacted concrete spillway - Raised crest of embankment - Installed a partial chimney drain seepage collection system - Installed wick drains - Installed new pipeline and downstream control structure
4. Insufficient Spillway Capacity Middle Branch Dam (MWH, 2000)	- Insufficient spillway capacity - Low level outlet works in need of rehabilitation	- New auxiliary spillway with a two-bay fuse plug - Added roller-compacted concrete to crest and downstream slope of embankment

TITLE – PROJECT - REFERENCE	PROBLEMS	SOLUTIONS
5. Foundation Liquefaction Wickiup Dam (Bliss, 2003)	- Foundation liquefaction	- Jet grouted of foundation - Added a blanket filter and drain system
6. High Underseepage Gradients Miami Conservancy District (MWH, 2005)	- High seepage gradients	- Add relief well system - Add downstream drainage collection system - Add passive weighting berm with underseepage collection and discharge system - Extended core walls
7. Seismic Instability Diversion Dam (Findlay & Rabasca, 2003)	- Hydraulic fill dam - Susceptibility to liquefaction during seismic event - Abutment seepage due to lack of core wall	- Added buttressing berm along toe - Added drainage collection and discharge system

TABLE D–4: TIMBER AND MASONRY DAMS – SECTION 4.3.6.C

TITLE – PROJECT - REFERENCE	PROBLEMS	SOLUTIONS
1. Timber Crib Dam Deterioration Wallowa Falls Project (Greenman *et al.*, 1997)	- Deterioration of timber dam	- Added a rockfill dam with central impervious core
2. Masonry Dam Inadequate Stability & Spillway Capacity Boyd's Corner Dam (Prendergast, 1991)	- Inadequate spillway capacity - Inadequate stability - Leakage	- Added an ogre-shaped spillway - Installed post-tensioned anchors - Installed a grout curtain
3. Timber Crib Dam Deterioration Centennial Mill Dam (Blanchette, 1996)	- Deterioration of timber dam - Increasing costs of flashboard replacement - Lost generation due to flashboard loss	- Constructed new concrete dam with post-tensioned rock anchors - Added a inflatable rubber dam to spillway crest

TABLE D–5: RESERVOIRS – SECTION 4.3.6.D

TITLE – PROJECT - REFERENCE	PROBLEMS	SOLUTIONS
1. Flood Design Criteria Change Upper Racquette River Project (Brookfield Power New York, 1989)	- Change in flood design criteria	- Performed reservoir routing study
2. Reservoir Liner Rehabilitation 3. Seneca Pumped Storage Station (Brzytwa and Finis, 2000)	- Foundation settlement - Performance of asphalt-concrete liner	- Installed new asphalt-concrete liner

TABLE D–6: POWERHOUSES – SECTION 4.4.4

TITLE – PROJECT - REFERENCE	PROBLEMS	SOLUTIONS
1. Deteriorated Concrete Spiral Case Wapato Indian Irrigation Project (USBR, 2000)	- Cracks - Leakage	- Waterproof coating
2. Powerhouse Instability (Flood Loads) Buck Hydroelectric Project (AEP, 1993)	- Instability under PMF condition	- Installed post-tensioned anchors
3. Powerhouse Instability (Uplift Pressures) Schaghticoke Project (Brookfield Power New York, 1999)	- Uplift pressures from two aquifers affecting stability of slopes, penstock, and powerhouse - Surface water runoff	- Install pressure relief wells with gravity collection system - Construct bench cuts and install underdrains along penstock slope

TITLE – PROJECT - REFERENCE	PROBLEMS	SOLUTIONS
4. Life Extension Study Cabot Project (Kleinschmidt, 1993)	- Life extension and continued operation - Canal surge - Antiquated rack and pinion gate operators - Intake gates not able to close fast enough - Soft concrete in turbine spiral case - Spiral case leakage - Plugging of powerhouse drainage system - Previous grouting programs - Structural cracking of powerhouse substructure	- Life extension study to evaluate options for upgrade or major modifications to project to allow continued operations - Mechanized and automated powerhouse spillway gates in the forebay - Canal intake gates automated and upgraded with hydraulic system - Modified operating procedures for canal intake gates - Grouted cracks and installed air vent holes in powerhouse - Grouted cracks in spiral case
5. Antiquated Bridge Crane Bonneville No.1 Powerhouse (USACE, 2000)	- Erratic and non-responsive controls - Excessive wear of Trolley wheels and rails - Crane skewing - Deteriorated wood supports - Asbestos and lead paint	- Replace crane - Replace electrical systems - Structural modifications to rail supports - Addressed asbestos and lead paint
6. Deteriorated Supports and Antiquated Bridge Crane Minidoka Project (USBR, 1992)	- Antiquated crane - Crushed timber crane rail supports	- Replace crane - New crane rail on grout pad

TABLE D–7: FISH PASSAGE – SECTION 4.5.5

TITLE – PROJECT - REFERENCE	PROBLEMS	SOLUTIONS
1. Energy Loss to Fish Attraction Flows McNary Dam (MWH, 1996)	- Need for energy dissipation device for fish attraction flows. - Loss of generation to attraction flows.	- Install hydroelectric turbine to discharge attraction flows.
2. Entrainment of Juvenile Fish Puntledge Project (BC Hydro, 1995)	- Entrainment fish mortality. - Bypass of juvenile migrants.	- Hydraulic model testing. - Install fish screens in penstock.
3. Outdated Fish Bypass System and Evaluation Facility. Bonneville Project (USACE, 2000)	- Sampling facilities were biological and technical deficient. - Fish transportation system was deficient due to channel velocities were out of criteria, insufficient water supply, high screening velocities, high outfall predation and mortality, and system delay.	- Construct modern sampling and testing facility. - Construct new flume to transport fish well below influences of the project.
4. Leaping Fish John Day Lock and Dam (USACE, 2000)	- Fish jumping from fish ladder. - Unfavorable hydraulic conditions.	- Model testing to assess hydraulic conditions and model corrections. - Redesign and replace ladder baffles.
5. Energy Loss to Fish Attraction Flows Woodland Project (Kleinschmidt, 1992)	- Loss of generation due to discharge of fishway attraction flows.	- Install a pump-back system to eliminate the discharge of supplementary attraction flows.
6. License Mandate for Fish Passage Greenville Dam (Kleinschmidt, 1996)	- Mandated installation of upstream and downstream fish passage facilities. - Cost of project development. - Size of fish run. - Increase in upstream flood stage.	- Install fish lift in place of fish ladder. - Computer models and negotiation on magnitude of flood impacts.

7. Dam Repair and Mandate for Fish Passage Rainbow Dam (Kleinschmidt, 1993)	- Need for repair of deteriorating concrete dam. - Need for stabilizing dam. - Obtaining agency permits to perform repair and stabilization work.	- Construct downstream fish passage facility. - Install overlay on trash racks to reduce fish entrainment. - Design and partial construction of fish sorting facility.

TABLE D–8: TRASH RACKS – SECTION 4.6.5

TITLE – PROJECT - REFERENCE	PROBLEMS	SOLUTIONS
1. Deteriorated Steel Trash Racks Berrien Springs (AEP, 1996)	- Deteriorated racks - Poor original installation	- New trash racks
2. Temporary Trash Rack Repair Oswego Falls East (Brookfield Power New York, 1999)	- Deteriorated and deformed racks - Unsafe conditions	- Installed temporary racks downstream of existing - New permanent replacement racks to be installed in future
3. Trash Rack Failure Due to Inadequate Design Barge Canal Hydraulic Race (Brookfield Power New York, 1998)	- Failure of rack supports - Entrainment of plastic pool	- New supports and racks designed for 100% blinding
4. Blockage by Ice and Zebra Mussels School Street Hydroelectric (Brookfield Power New York, 1990)	- Build-up of frazil ice - Build-up of zebra mussels	- Install plastic racks

TITLE – PROJECT - REFERENCE	PROBLEMS	SOLUTIONS
5. Trashrack Failure Due to Inadequate Design (Pump Turbines) Smith Mountain (AEP, 1967)	- Inadequate understanding of hydraulic forces acting on structure	- Installed a movable trash rack system

TABLE D–9: TRASH RAKES – SECTION 4.7.5

TITLE – PROJECT - REFERENCE	PROBLEMS	SOLUTIONS
1. Debris Handling Racine Project (AEP, 2001)	- High labor costs - High debris disposal costs - Lost generation due to restricted turbine operation during trash raking	- Installed an automated trash rake system of the hoist and carriage type
2. Antiquated Trash Rake Byllesby Project (AEP, 1997)	- Labor intensive trash rake - Rake could not remove debris fast enough - Build-up of sediment and debris upstream of racks	- Installed an automated trash rake system of the dragline type
3. Debris Handing Dalles North Shore (USACE, 2001)	- Labor intensive raking - Concerns for safety of personnel - Raking performed only in daylight hours	- Installed an automated trash rake system of the hydraulic telescoping boom type

TABLE D–10: INTAKE CANALS, FLUMES AND FOREBAYS – SECTION 5.2.5

TITLE – PROJECT - REFERENCE	PROBLEMS	SOLUTIONS
1. Emergency Breach of Canal Embankment (Flood Condition) Constantine Project (AEP, 1989)	- Inability to lower head gates - Possible overtopping of embankments	- Breach an embankment - Rebuild embankment for low area for dedicated breach
2. Ice Induced Failure of Forebay Skimmer Wall Safe Harbor Project (MWH, 1999)	- Formation of natural ice dam - Ice forces against and under existing skimmer wall - Failure of skimmer wall	- New skimmer wall with vertically stacked stoplogs designed to fail under extreme loading
3. Deterioration of Water Conveyance System Naches Project (deRubertis, 1996)	- Deterioration of canal embankments - Failure of embankments due to condition - Root damaged of concrete lined canal - Deteriorated concrete liner - Deteriorated concrete flumes - Deteriorated concrete walls in forebay	- Repair features based on need/risk of failure - Replace conventionally cast concrete elevated flumes in kind - Replacement of forebay spill walls in kind - Canal lining repairs - Strengthen canal embankments
4. Upgrade of Forebay and Pipeline Santa Ana River Hydroelectric Project (MWH, 2001)	- Flood control dam inundates existing project - Fault crossing tunnel alignment	- Replace flumes with penstocks - Oversized culvert placed around pipeline crossing fault - Abandon one powerhouse
5. Plant Expansion and New Forebay Dam Grand Coulee Hydroelectric Project (USBR, 1981	- Faults and major joints in foundations - Forebay dam to be constructed on bench above powerhouse	- Extensive sub-surface exploration and laboratory testing program - Finite element models to determine stability of dam and foundation

TABLE D–11: TUNNELS, SHAFTS AND UNDERGROUND OPENINGS – SECTION 5.3.5

TITLE – PROJECT - REFERENCE	PROBLEMS	SOLUTIONS
1. Structural Instability of Tunnel Terror Lake Project (de Rubertis, 2001)	- Rockfall in shaft	- Install steel liner and backfill space behind liner with concrete
2. Leakage of Concrete Tunnel Liner Packwood Project (Passage, 2002)	- Cracked concrete liner - Heaved invert slab - Leakage	- Install membrane to tunnel perimeter - Install new invert slab - Grout voids
3. Inadequate Investigation of Subsurface Conditions Power Creek Project (de Rubertis, 2002)	- Tunnel through closely jointed and faulted rock	- Install permanent support system - Install shotcrete liner
4. Fault Zones and Deteriorated Concrete Liner Spirit Lake Project (USACE, 1998)	- Fault zones in rock - Deteriorate shotcrete lining - Damaged support ribs	- Installed new support ribs - Install replacement concrete and shotcrete
5. Deteriorated Wood Pipeline (Replacement) Trenton Project (Brookfield Power New York, 1982)	- Wood pipeline reached the end of service life	- Replaced with combination tunnel and above ground steel pipeline
6. Tunnel Surge Roberts Tunnel Hydro (Denver Water, 1995)	- Excessive vibration of power generating equipment - Surging due to equipment operation - Failure of turbine thrust bearings	- Operate equipment to suit hydraulic limitations of tunnel

TABLE D–12: PENSTOCKS – SECTION 5.4.5

TITLE – PROJECT - REFERENCE	PROBLEMS	SOLUTIONS
1. Wood Stave Penstock Deterioration (Replacement) Bigfork Project (Pacificorp, 2001)	- Wood penstock reaching end of service life - Penstock failure	- Partial life extension - Replace with polyethylene pipe
2. Wood Stave Penstock Deterioration (Replacement) Victoria Hydro Project (MWH, 2001)	- Advanced deterioration of wood staves - Ovaling - Settlement - Leakage	- Replace with above ground steel penstock
3. Steel Penstock Deterioration and Failure Schaghticoke Project (Brookfield Power New York, 2000)	- Failure caused by localized corrosion	- In kind replacement of penstock and surge tank
4. Foundation Movement Camino Project (MWH, 2000)	- Toppling and sliding of bedrock - Downslope movement of penstock	- Mass grouting of rock slope - Anchoring of penstock
5. Deterioration and Increased Headloss Kingston Mills Project (EOP, 1995)	- Diminished thickness of penstock walls - Zebra mussel	- Clean with high pressure soda blast - Apply high pressure paint system

TABLE D–13: TAILRACES – SECTION 5.5.5

TITLE – PROJECT - REFERENCE	PROBLEMS	SOLUTIONS
1. Low Seasonal Tailwater Level NY Barge Canal – Lower Mechanicville Hydroelectric Facility (NYSDEC, 1996)	- Turbine operation impacted by low tailwater levels, impacting	- Install Jersey barrier to seasonally raise tailwater level
2. Flood Erosion of Tailrace Embankment Combie Project (de Rubertis, 1999)	- Erosion of tailrace embankment	- Install grouted riprap

TITLE – PROJECT - REFERENCE	PROBLEMS	SOLUTIONS
3. Deteriorated Training Wall Yaleville Project (Brookfield Power New York, 1982)	- Deterioration of concrete training wall	- Do nothing
4. High Tailwater Level New Colgate Powerhouse (Yuba CWA, 2003)	- Irregular runner rotation, excessive turbine vibration, and instability of power output due to high tailwater levels during flooding	- Install an automated compressed air tailwater suppression system

TABLE D–14: GATES AND GATE HOISTS – SECTION 6.2.5

TITLE – PROJECT - REFERENCE	PROBLEMS	SOLUTIONS
1. Poor Sealing and Gate Closure Spier Falls Project (Brookfield Power New York, 1992)	- Badly pitted seal plates - Broken or missing roller chains and guides - Gate not closing and sealing properly	- Replace seal plates, roller chain guides, and roller chains with corrosion resistant materials - Adjust set screws on bottom seal
2. Tainter Gate Leakage, Freezing, and Concrete Deterioration Heuvelton Project (Brookfield Power New York, 1994)	- Ice buildup due to seal leakage and precipitation - Deteriorated concrete structure	- Rebuild concrete structure - Install rubber dams in place of two tainter gates
3. Trunnion Pin Failure Dalles Project (USACE, 2001)	- Damaged trunnion pin due to shear-off f keeper plate bolts - Lubricant not reaching trunnion bushing	- Replace trunnion bushing with self-lubricating material - Install stainless steel trunnion pins
4. No Provisions For Maintenance Bulkheads Reusens Project (AEP, 1998)	- Need for general gate repairs - No means to dewater gates	- Provide a floating bulkhead system

TITLE – PROJECT - REFERENCE	PROBLEMS	SOLUTIONS
5. Excessive Movement and Tainter Gate Failure Elkhart Project (AEP, 1997)	- Failure of tainter gate arm - Increase in sliding friction - Excessive gate movement	- Replace both gate arms - Realign the gate
6. Insufficient Gate Capacity and Loss of Generation Byllesby Project (AEP, 1997)	- Lost generation due to loss of stoplog (stanchion type)	- Install inflatable rubber gate
7. Uncontrolled Flashboard Failures Allens Falls Project (Brookfield Power New York, 1991)	- Uncontrolled flashboard failures	- Install bottom hinged crest gates
8. Safety Concerns with Antiquated Gate Operator Belfort Hydroelectric (Brookfield Power New York, 1997)	- Broken gear teeth - Physically demanding and dangerous to use - Gate operator had exceeded its useful service life	- Installed new lifting frame with overhead hoist
9. Sector Gate Rehabilitation Post Falls Hydroelectric Development (MWH, 2004)	- Deteriorating and aging gate - Aging gate operators - Lack of means to dewater gate - Deteriorated concrete - Unknown and previous modifications to concrete structures	- Rebuild and rehabilitate gate and gate operators - Innovative and flexible dewatering system - Assemble historic data and field confirmation of structure - Detailed underwater inspections

TABLE D–15: VALVES AND OPERATORS – SECTION 6.3.5

TITLE – PROJECT - REFERENCE	PROBLEMS	SOLUTIONS
1. Safety and Operational Concerns Carolina Power & Light (Kleinschmidt, 1997)	- Lanier-Johnson Valve - Deteriorated sealing surfaces - Inability to control motion of valve - Cracked valve body - Susceptibility to slamming closures - Concerns with personnel and plant safety	- Replace with spherical valve

TABLE D–16: FLASHBOARDS – SECTION 6.4.4

TITLE – PROJECT - REFERENCE	PROBLEMS	SOLUTIONS
1. Flashboard Operation Misunderstanding Gore Mountain Dam (NYSDEC, 1989)	- Filed modifications of flashboards - Misunderstanding of flashboards design and operation	- Redesign flashboards - Educate operating personnel
2. Flashboard Failure Due to Ice and High Flows Deferiet Project (Brookfield Power New York, 2000)	- Excessive loss of generation and generating capacity due to annual loss of flashboards	- Replace with inflatable flashboards (rubber dam)
3. Personnel Safety, Lost Generation and O&M Costs Anson Project (Kleinschmidt, 1997)	- Lost generation - Costs to replace and maintain flashboards - Safety of dam personnel	- Replace with inflatable rubber dam
4. Insufficient Spillway Capacity and Loss of Flashboards Shelburne Dam (Kleinschmidt, 1990)	- Insufficient (gated) spillway capacity - Replacement cost of flashboards - Lost generation - Deterioration of timber crib sill	- Rebuild sill with concrete - Replaced part of flashboards with bottom hinged crest gates
5. Over-Design of Flashboards and Failure of Spillway Crest Lower Pelzer Project (Kleinschmidt, 1996)	- Over-designed flashboards - Failure of spillway crest	- Rebuild crest with concrete - Redesign flashboard system to fail, or release manually

Index