



# Manual for Assessing Hydraulic Safety of Existing Dams - Volume I

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Central Water Commission  
Ministry of Jal Shakti  
Department of Water Resources,  
River Development & Ganga Rejuvenation  
Government of India

**Front Cover Photograph:** Sardar Sarovar Dam (India) during the monsoon flood.



Government of India  
Central Water Commission  
Central Dam Safety Organization

# Manual for Assessing Hydraulic Safety of Existing Dams

**June 2021**

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Disclaimer

The Central Water Commission under the Dam Safety and Improvement Project has undertaken to prepare this *Manual for Assessing Hydraulic Safety of Existing Dams* to provide necessary guidance for ensuring the safety of existing dams against adverse hydrologic and hydraulic events. The design studies and measures required will vary from dam to dam depending on the type of problems encountered. While every effort has been taken to incorporate all basic details as per the latest state of the art, yet it is not possible to cover all the conditions/problems which may be faced in the field. CWC absolves itself from any responsibility in this regard and dam owners and others involved with the dam rehabilitation activity should use their discretion in implementing the guidelines contained in this Manual.

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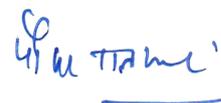
## MESSAGE

The Central Water Commission has been publishing various Guidelines and Manuals in the area of dam safety under the World Bank assisted Dam Rehabilitation and Improvement Project. This Manual titled '*Manual for Assessing Hydraulic Safety of Existing Dams*' is intended to provide dam safety professionals with wealth of information and references on various aspects related to the hydraulic safety of existing dams, reservoirs and their appurtenant works, aiming to ensure safety of the dam and minimizing the risk to which the downstream population may be subject to, due to any dam incidents or failures.

Hydraulic Safety of existing dams is an important concern amongst the dam owning agencies. This publication presents detailed aspects regarding assessment, evaluation and rehabilitation of hydraulic structures including spillways, outlet works and energy dissipation arrangements in existing dams. The document provides information for identifying and dealing with typical hazards related to hydraulic functioning. A number of case studies and failure modes catalogue are included in order to facilitate in a better understanding of the mechanics associated with various hydraulic problems/issues.

This Manual is expected to assist dam engineering community for managing the safety of dam-reservoir systems during their operative life. The procedures, techniques and measures prescribed through various chapters of this document follow contemporary global best practices related to review of hydraulic safety of an existing dam and associated appurtenant structures both for the existing design flood as well as for the revised design flood.

I hope, this Manual can be used as an excellent reference material by our engineers/dam owners while carrying out comprehensive hydraulic safety review and appropriate planning for rehabilitation of their dams. The contents of this publication is well organized and would provide readers with clear understanding of hydraulic problems in dams and the possible remedial measures for improving the health and safety of dams in a systematic manner.



(S K Haldar)  
Chairman

Central Water Commission

New Delhi  
June 2021

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## FOREWORD

The existing dams need a sound hydraulic safety assessment in order to work out rehabilitation measures. The hydraulic design and performance is required to be reviewed for the existing dams in case revised design flood has increased significantly as a result of periodic safety reviews. It is often seen that there is a considerable upward revision in design flood in a large number of dams. Some dams may get overtopped owing to substantial increase in revised design floods.

This Manual titled '*Manual for Assessing Hydraulic Safety of Existing Dams*' deals with the hydraulic safety of all the components of the system viz. dam and reservoir, approach channel, different types of spillways, their control and conveyance structures, outlet works in earthen/masonry/concrete dams, their intake and conveyance structure, hydro-mechanical equipment, different types of energy dissipation arrangements, plunge pool and exit channel. Various hazards and their adverse response/effects on different components have been discussed under this document.

The hazards are in general due to increase in design flood resulting in increase in MWL, reservoir sedimentation, obstructions due to floating debris/ice, inoperative gates, condition of concrete surface in conveyance structure and energy dissipater, effects of increase in discharge over spillway and energy dissipater, limitations in exit channel capacity, erosion of bed and banks of exit channel, etc. The adverse response/effects include lack of spillway capacity, dam overtopping, temporal loss of gates, lack of conveyance capacity, overtopping of chute/energy dissipater walls, damages due to abrasion/cavitation, increase in hydraulic and hydrodynamic loads in spillway and energy dissipater, malfunctioning of energy dissipater, instability of hydraulic jumps etc.

For increasing the spillway capacity in addition to additional conventional spillways, the comparatively recent options of unconventional spillways which include fuse plug, labyrinth spillway, piano key spillway, fuse gates, fuse plug (concrete blocks), open channels (flush bars), overtopping of dam with protected section and stepped spillways have been discussed. Under energy dissipation various rehabilitation measures for increasing the capacity of terminal structures in spillways have been comprehensively covered.

I hope that professionals engaged in the comprehensive safety review of dams will find this Manual very useful in managing the hydraulic safety aspects. I thank and compliment all the individuals who have contributed to the preparation of this Manual and hope that the efforts will go a long way in improving the dam safety environment in the country. Central Water Commission also acknowledges the special support given by all members of Review Committee in finalizing this Manual.

New Delhi  
June 2021



( Dr. R K Gupta )  
Member (D&R)  
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## PREFACE

This Manual provides access to the state-of-the-art information and references on various aspects related to the hydraulic safety of existing dams. A number of case studies and failure modes catalogue are included in order to facilitate in a better understanding of the various hydraulic problems which may lead to either partial or total failure of the works. All the documentation included is expected to be helpful in evaluation of the safety levels of the dam, spillway, outlet works, energy dissipation arrangements and other appurtenance works and in working out options for rehabilitation from the view point of Hydraulic safety.

The Volume 1 of the Manual contains five chapters namely; Introduction, Dam and its Reservoir, Spillway, Outlet Works and Energy Dissipators. Each chapter contains an overview of the subject/component, description of the component, assessment of hydraulic safety of that component on account of various possible hazards/defects, rehabilitation measures which can be undertaken and lessons learnt along with some case studies for illustration.

Some recent developments covered in the Manual are Overtopping protection for dams and different types of unconventional spillways. Introduction to risk analysis by identification of possible failure modes is also discussed. Various examples/case studies of documented cases included in the Manual are expected to help/contribute in understanding of the issues/problems related to Hydraulic Safety of a Dam-Reservoir System and its elements in right context.

The Volume 2 of the Manual has six appendices.

Appendix A represents an approach to the process of Risk Assessment and Management, which envisages identification of failure modes. More and less likely factors which could lead to development of a particular failure mode have been brought out. Also, an array of suggested actions has been included.

Appendix B contains few case studies involving various types of incidents/failures in dams. This appendix contains failure of Upper dam (CFRD) of Taum Sauk Pumped Storage Hydro-Electric project, USA which was due to excessive pumping from the lower reservoir and failure of instrumentation systems leading to overtopping. The case study on Spencer Dam, USA is perhaps the first of a dam to fail by overtopping due to blockage of spillway gates by ice. El Guapo dam failure, Venezuela envisages a different failure mode not by overtopping over the dam section but by overtopping of chute walls and energy dissipation. A number of incidents are presented in detail on Lower Caroni Cascade Hydroelectric Development, Venezuela with an installed capacity of 15830 MW (Guri, Macagua and Caruachi dams) and yet to be completed 2220 MW Tocoma dam. Maneri dam in India is a case study illustrating damages due to abrasion over the spillway on account of high suspended load in flowing water and rolling boulders. The last case study is the failure of a gate at Narayanpur dam in India.

Appendix C deals with retention of debris by means of floating barriers (booms).

Appendix D contains description on state-of-the-art Hydraulic Modeling. It covers both Physical and Numerical Modeling.

Appendix E covers the Operational safety of Hydro-mechanical equipments which constitutes an important aspect for the Hydraulic safety of gated spillways. Finally, the

Appendix F covers the Glossary of terms for dam safety.

This Manual is expected to aid the engineers who are responsible for reviewing the hydraulic safety of dams in order to plan, design & construct various rehabilitation works in a comprehensive way

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## Chapter 1. INTRODUCTION

In recent years many unprecedented and extremely severe floods have been witnessed both in India and in other countries. For example, in August 2019, devastating floods hit the state of Kerala, for the second year in a row. The floods caused millions to flee from their homes and an extensive loss of life and property. The rainfall recorded during the recent storm of 15-17 August 2018 was close to the highest ever recorded during Devikulam storm of 16-18 July 1924 in that region.

Scientists suspect that these extreme events may also be linked to climate change. As such their probability of occurrence is likely to increase with time. These flood events in turn also impact on the hydrologic and hydraulic safety of existing dams.

About 25% of all large dams in India are at least 50 years of age. More than 50% are older than 40 years. Further more than 60% of India's large dams that are still in active use are over 120 years old. Aging and poor maintenance have been some of the causes of malfunctioning of hydro-mechanical equipment and poor and deteriorating conditions of the dams; there is also a need of addressing the hydrological deficiencies in many of the dams. Further a few dams have also encountered some dam safety related incidents like dam slope failures, gate failures etc. In addition their reservoir capacity is decreasing with time due to reservoir sedimentation.

It is well known that dams and their reservoirs provide a number of benefits to society such as water-supply, irrigation, hydro-power, flood control, recreation etc. To protect the dam as well as the population and properties downstream of each dam, Dam Safety Reviews are required periodically to address issues related to seismic and extreme flood events besides many other urgent issues which may be faced by the dam. Efforts

should be focused on improvements to the existing dams such as modification of existing spillways and/or addition of new auxiliary/emergency spillways to cater to the revised inflow design floods. These are to be simultaneously accompanied by rehabilitation/repairs of other associated structures such as control gates, spillway, energy dissipaters etc. This Manual attempts to cover the hydraulic aspects related to dam safety

### 1.1 Purpose

The purpose of this Manual is to provide engineers and dam owners, access to the state of the art information and references on aspects related to the hydraulic safety of existing dams, reservoirs and their appurtenant works, aiming to provide assistance in taking care of the dam safety and at minimizing the risk to which the downstream population may be subject to, due to any dam incidents or failure of the dam. The Manual attempts to provide the most pertinent information for identifying and dealing with typical hazards related to hydraulic functioning. A number of case studies and a failures modes catalogue, are included in order to facilitate the reader in a better understanding of the mechanics associated with various problems which may lead to damages either partial or total or failure of the works. All the documentation included in this Manual is expected to be helpful in evaluation of the safety levels of the dam, reservoir system and its appurtenances works, in identification of the actual or potential adverse responses of structures to expected hydraulic actions and in working out the options for rehabilitation.

For the purpose of this manual, Hazard is defined as the mechanism that triggers the development of a failure.

## 1.2 Overview of Hydraulic Safety of Dams

### 1.2.1 Dam Safety: Structural and Hydraulic safety

Central Water Commission (CWC) has been advising the dams owners, on matters of Dam Safety Management in India, since long. It has been focusing in producing guidelines and manuals on different aspects related to the safety of dams, which will be useful references. Further the need to guarantee, recover or extend the operational life of the existing India's dam park is an important requirement.

The definition of dam also includes its abutments, appurtenant works and the impounded water. Therefore, dam safety encompasses all hydraulic works such as the reservoir/lake and its rim, the dam, the appurtenant works (spillways and outlet works), and structures close to the dam, watercourse/river, and abutments. Thus, if security of these works is not guaranteed *then there could be different kinds of incidents*, not restricted to: (1) Overtopping of the dam due to several causes (with or without failure) , (2) Undermining (piping) in a dam and increase in seepage with time (3) Slope failure of a dam (4) Blockage of the spillway due to floating material/debris or Mal-functioning of the spillway gates etc.

Hydraulic Safety of Dams encompasses both *“physical safety of the dam itself”* and *“operational safety”*, in the event of a flood. Thus, the works, in this case - the dam and its appurtenant works, must retain their physical integrity, stability and resistance to safely withstand all the forces acting on it for all conditions of loading; and also, they must have reliable hydraulic performance. Since security means “almost at any moment”, safety should be verified in both normal and extreme conditions.

Soriano (2008), states: *“Dam safety is a margin that would separate the real conditions that exist in*

*the built dam from those that lead to its destruction or deterioration”*. The conditions referred to above are structural and hydraulic; so, both must be considered simultaneously. Further *“a stable and resistant dam whose appurtenances works are malfunctioning, is not a safe dam”*. In addition, “real conditions” encompasses not only basic aspects such as the capacity and resistance of the dam and spillway to withstand all possible loads but also aging, material fatigue, obsolescence, operational and maintenance history and many others that reduces dam safety. Internationally, “Structural Safety” and “Hydraulic Safety”, together constitute the safety of a dam.

Soriano and Escuder (2008) defines dam safety as *“The suitability of operation of spillways and outlet works, in an ample range of discharges, including ordinary and extraordinary flows”*. Under this criterion, failure of these hydraulic structures can be due to: (1) Lack of hydraulic capacity of the spillway arrangements to safely pass the design flood, (2) Lack of enough structural capacity of the dam and spillway to resist loads under different conditions and (3) Failure in re-integration of the flood discharges flowing down the spillway/outlets with the downstream watercourse/river in appropriate conditions.

Hydraulic safety is often included in hydrological safety of the dam and/or reservoir; other authors add both by way of hydrological-hydraulic safety. Hydrological safety focuses in extreme flood event occurrences and the responses of the dam-reservoir system. Hydraulic safety, as mentioned earlier, applies to functioning of various components of the dam & reservoir, spillways, outlet works, energy dissipaters etc. (Civil works, Hydro-Mechanical, and Electrical equipment, and others) during both normal and extreme events. There is an overlapping zone between them (i.e. Hydrologic safety and Hydraulic safety) that corresponds to routing of the flood through the reservoir and its management to decide the adequacy of the spillway capacity and freeboard for the dam. These aspects are discussed in the Guideline *“Selecting and Accommodating Inflow*

*Design Flood for Dams*” (CWC, 2021) and *“Manual for Assessing Structural Safety of Existing Dams”* (CWC, 2020).

Another important aspect of Hydraulic safety is related to the safety of the downstream areas and the population, utilities and infrastructure at risk during an extreme event of dam break and/or release of uncontrolled discharge. This aspect is however based on occurrence of failure of the dam and then routing of the generated flood wave along the river. Even though vulnerability of downstream zone and consequences of dam break are considered to define hazard potential of the dam, its management is commonly carried out through emergency action plans and/or specific structural and non-structural measures. This aspect of hydraulic safety is not included in this Manual but is covered in the Guidelines for developing Emergency Action Plan for dams prepared by CWC (CWC, 2020).

The present Manual deals with assessing hydraulic safety of the dam-reservoir system; in other words, it deals with the “Safe passing of floods”. It refers to assessing safety of various structures i.e. dam & reservoir, spillway and outlet works and associated components of control, conveyance and energy dissipation and downstream channel/river from hydraulic considerations. This Manual is a complement of the *“Manual for Assessing Structural Safety of Existing Dams”* (CWC, 2020); thus, both manuals are inter-related.

### 1.2.2 Classification of dams

For assessing hydraulic safety of dams, the first step is the classification of the dams according to various parameters which are related with the impact of its failure as dam break or significant damage of works or mal-

functioning of any component.

Presently there are two approaches related to classification of dams in India:

- The current approach to classify dams in India based on storage and head (IS 11223)
- The proposed dam classification system as per the new guidelines on Hazard potential of dams, (CWC, 2020)

In this context the following information is relevant:

- a) Large dam vs. Small dam. In the text of “The Dam Safety Bill 2019” (Ministry of Water Resources, India), in process of enactment, a large dam is defined as: “(i) Dams above fifteen meters in height, measured from the lowest portion of the general foundation area to the top of dam; or (ii) Dams between ten meters to fifteen meters in height and satisfies at least one of the following, namely: (A) the length of crest is not less than five hundred meters; or (B) the capacity of the reservoir formed by the dam is not less than one million cubic meters; or (C) the maximum flood discharge dealt with by the dam is not less than two thousand cubic meters per second; or (D) the dam has especially difficult foundation problems; or (E) the dam is of unusual design”.
- b) IS: 11223-1985 presents a dam classification that considers hydraulic head and reservoir storage at FRL ( Full Reservoir Level) , as shown in Table 1-1.
- c) Dam hazard classification as in Central Dam Safety Organization (CDSO)-CWC publication (1987) is

Class of dam	Static head (m)	Gross reservoir's storage (Mm <sup>3</sup> )	Inflow design flood for safety of dam:
SMALL	12 or less	10 or less	100-year flood
INTERMEDIATE	12 to 30	10 to 60	SPF
LARGE	30 or more	60 or more	PMF

Table 1-1: Classification of Dams in India (IS: 11223-1985)

as under:

- High hazard dam: A dam whose failure would cause the loss of life and severe damage to homes, industrial and commercial buildings, public utilities, major highways, or railroads.
  - Significant hazard dam: A dam whose failure would damage isolated homes and highways, or cause the temporary interruption of public utility services.
  - Low hazard dam: A dam whose failure would damage farm buildings, agricultural land, or local roads.
- d) CWC is planning to come out with the following two guidelines to define the IDF to be used in existing dams namely “Guidelines for Selecting and Accommodating Inflow Design Flood for Dams” and "Guidelines for Classifying the Hazard Potential of Dams " (CWC, 2020).

However, at present IS 11223 is being used.

- e) The Guidelines for Classifying Hazard Potential for Dams (under preparation) contemplate to classify dams in the Indian context based on consequences of dam failure, as per the current international trends and according to the status of development of risk management of dams in the country. This system uses four classes of potential hazard due to dam failure, instead of three considered according to CDSO (Central Dam Safety Organization) classification.

The dam classification based on potential hazards is proposed by means of the Potential Consequence Index of the dam (as explained in the “Guidelines for Classifying the Hazard Potential of Dams”. Table 1-2 presents a summary table of the proposed Dam Classification as per hazard potential.

Hazard Potential Class	Potential Consequences Index ( $P_{CI}$ )	Consequences Categories			
		Capital Value of Project	Potential for Loss of Life	Potential for Property Damage	Potential for Environmental and Cultural Impact
Class I	<300	Low	None. Temporal or no incremental population at risk, no potential loss of life is expected. No inhabited structures	Minimal. Limited economic and agricultural development	None
Class II	<300	Average	Minimal or low population at risk. No potential loss of life is expected even during the worst-case scenario of emergency management	Notable agriculture or economic activities. States highways and/or rail lines	Minimal incremental damage. Short-Term or reversible impact (less than 2 years)
Class III	$300 < P_{CI} < 600$	Significant	Considerable. Several inhabited developments. Potential for loss of life highly dependent of the adequacy of warning and rescue operations.	Significant industry, commercial and economic developments. National and state highways and rail lines	Limited. Impact have a midterm duration (less than 10 years) with high probability of total recovery after mitigation measures
Class IV	>600	Critical	Extreme. High density populated areas, Potential for loss of life is too high even during the best scenario of emergency management.	Highly developed area in terms of industry, property, transportation and lifeline features	Severe. Long-term impact/effects in the protected areas or cultural heritage sites with low probability of recovery.

Table 1-2: Dam classification as per hazard potential as recently proposed by CWC (CWC, 2020)

### 1.2.3 Rehabilitation and Dam safety

This Manual, as well as other Guidelines and Manuals are part of the publications being brought out on dam safety by CWC.

As per statistics available, over 60% of the failures in India have occurred in the first 10 years of operation, the most frequent cause being overtopping of earth dams due to floods. Figures 1-1 and 1-2 summarize the history and causes of dam failures in India. In addition, there are other issues which include malfunction of dams due to the aging of works, the absence of sustained monitoring and maintenance program and non-availability of sufficient funds for repair works. In this scenario, rehabilitation represents an activity of great importance to safeguard the safety of existing dams and its appurtenant works in the country.

An appurtenant work is rehabilitated to:

- Bring it to its initial operating condition (according to design and purpose originally assigned), with regard to its strength, stability and hydraulic behavior.
- Upgrade it to the new engineering criteria and practices (ICOLD, Indian Standards, others) in case of non-compliance with current standards.
- Adapt it to or expand its capacity to withstand new or greater actions

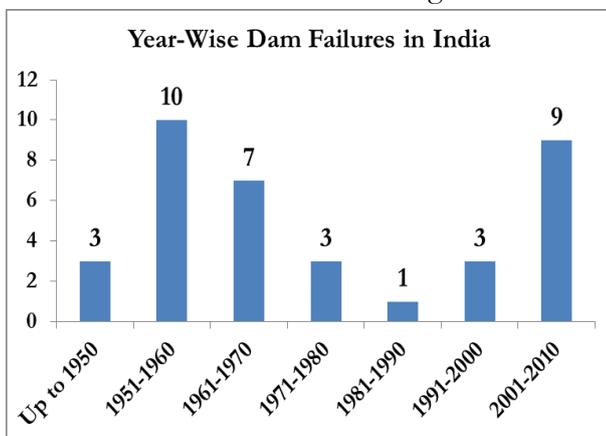


Figure 1-1: History of dam failures in India (DRIP, 2018). (loads). In case of hydraulic actions, a

typical problem faced is the requirement to pass larger floods resulting from a recent hydrological study considering the changes in the physical and hydrological characteristics of the catchment.

- Improve it by increasing its structural stability, durability etc.
- Improve it by removing and replacing equipment or installations (e.g. hydro-mechanical and electro-mechanical) due to aging, obsolescence, safety, operational ease or use of new technology (state of art equipment, mechanisms and controls), etc.
- Attend the requirements and results of the Dam Risk Analysis, defining the corrective action to manage risk to the expected level.
- Improve its facilities and civil components to reduce threats of anthropic origin, avoiding improper operation or neglect: ease and guarantee access of persons, tools and equipment, adequacy of space for operation and maintenance, signaling and upgrading electro-mechanical components, control and diversification of the electricity supply, PLC systems, others.

First of all an assessment of the physical condition of the works is required to be made for which a sound inspection and a technical audit of the dam-reservoir system

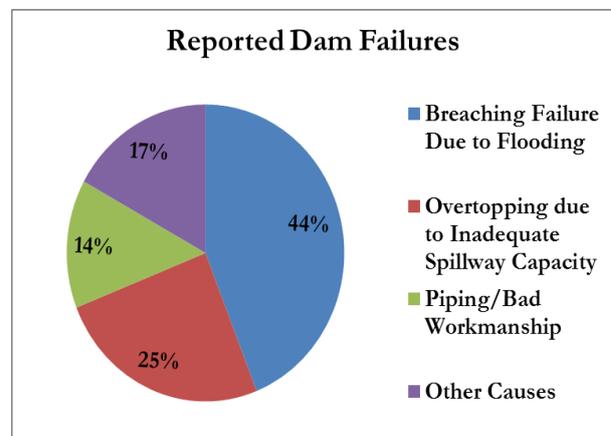


Figure 1-2: Causes of dam failures in India (DRIP, 2018).

is needed. This phase of the rehabilitation plan is covered in the publication: “Guidelines for Safety Inspection of Dams” (CWC, 2019).

There after the three main activities which are required to be carried out before undertaking rehabilitation measures are: (1) To define the cause or trigger event of the problem, (2) To carry out needful investigations and analysis to study the process or phenomenon and (3) Work out the cost of rehabilitation works required for repair of the damages and/or its progression due to malfunction or failure.

Further, although some cases of malfunction or damage may appear as an incident (a problem without failure) in the dam-reservoir system, its monitoring and rehabilitation may prevent any progressive or cumulative development, which can decrease the risk of failure (dam break and/or loss of performance) of the component, the appurtenant work and/or the dam.

The rehabilitation works to improve hydraulic security may not only include major structural solutions or local repairs or replacement of hydraulic equipment, but also could include complementary non-structural measures. Non-structural measures may include changes in operation rules, instrumentation, flood warning systems, training

of personnel, etc.

The scope of rehabilitation works, associated with hydraulic safety, will be the core of the Manual. This document presents how to address and manage the rehabilitation process of appurtenant works to take them to a defined level of hydraulic safety or to a tolerable risk or to their best possible condition both physical and functional. Thus, this process, which is a link of the general safety chain of the dam, encompasses auditing, visualization and analysis of failure modes, working out corrective measures and proposals for implementation of the improvements or extensions necessary for each component to ensure hydraulic safety. Figure 1-3 shows examples of rehabilitation measures being carried out in a spillway and an outlet work.

### 1.3 Scope and Objectives

This Manual intends to synthesize the state of art in aspects related to assessing the Hydraulic Safety of existing Dams in India. The Manual provides information to investigate and to evaluate Hydraulic Safety, propose rehabilitation measures for the appurtenant structure, the approach channel, intake, spillway, bottom outlet, energy dissipaters and incorporation of flow to the river channel.



Figure 1-3: Examples of rehabilitation of appurtenant works

Objectives of the Manual, include (but are not limited to) the following:

- Identification of the hydrological/hydraulic deficiencies in the dam and its appurtenant works in the context of dam safety.
- Introduce a methodology that allows the engineers involved in the Dam Safety process to assess the hydraulic safety of the dam, identifying vulnerabilities and associated failure modes of hydraulic origin and/or in the hydraulic elements of the dam
- Select alternatives of rehabilitation measures that reduce/eliminate the detected vulnerabilities or loss of safety in any component and to provide sufficient information and knowledge to initiate the design for the rehabilitation work selected option.
- Learn from other's lessons from historical incidents, causes, consequences, adopted rehabilitation measures and even post construction evaluation of outcomes.

#### **1.4 What is this Manual and how to use it?**

The Manual for Assessing the Hydraulic Safety of Existing Dams is intended to provide useful technical information to engineers involved in dam engineering for managing the safety of the dam-reservoir systems during their operative life. It assists in assessing the hydraulic safety of the appurtenant works subject to various hydraulic actions that may lead any of its components to an unsafe condition, representing malfunction, potential damage or even failure of the dam.

The term failure as used in this Manual broadly falls under the following two categories:

- 1- Functional failure of the spillway that leads to overflow and dam break with

uncontrolled discharge of water downstream, and

- 2- Failure of a component of the appurtenant work that involves temporary loss of its function and therefore, it represents an operational failure of the reservoir, generating a critical situation in terms of lack of protection against hydrological or operational events.

The topics of Hydraulic Safety are approached in a way so as to be applicable to the diverse types of appurtenant works, with focus on the "Safe passing of floods".

Contents of the Manual covers all major components from the reservoir, through the structures, to the receiving water body. The procedures, techniques and measures of rehabilitation of appurtenant works reproduced herein, follow most well-known international practices and represent the state of the art of dam hydraulics engineering. BIS issued by Indian Government as applicable are cited and mentioned along the text of the different chapters as the different topics are treated. At the end of the list of References for every Chapter, a list of BIS related to each particular theme is reproduced.

This Manual has the following contents:

Volume 1, consists of 5 Chapters and

Volume 2, consists of 6 Appendix:

In Chapter 1, Volume 1 an introduction to the way the subject of Hydraulic Safety of Existing Dams is planned to be dealt with in this Manual has been given along with contents of its Volume 1 & 2 and how they are connected. Orientation is given graphically to the reader to look for the information related to a particular hazard, their consequences, in a given location of the dam – reservoir system. Chapter 1 also presents the connection of the Manual with other Guidelines and Handbooks, written by CWC, as part of the DRIP Project.

Chapters 2 to 5 presents the sequence of structure definition – types – elements - hazard description – evaluation – experiences - rehabilitation measures for most frequently used structures in Indian dams.

The names of the chapters are:

Chapter 2- Dam and its Reservoir

Chapter 3- Spillways

Chapter 4- Outlets

Chapter 5- Energy Dissipators

Volume 2, portraits useful appendices:

- Appendix A includes a catalogue of Failure Modes Identification inherent to the Hydraulics Safety of Dams as treated in the Manual.
- Appendix B contains prominent case studies of dam failures, incidents, studies, investigations carried out, and the rehabilitation measures under taken.

- Appendix C deals with retention of debris by means of floating barriers (booms).
- Appendix D is a compendium on present international practices on Hydraulic Modeling of Dam- reservoir River systems
- Appendix E, contains a summary of aspects of Operational Safety of Hydro mechanical Equipment, causes of failure, and recommendations for risk reduction along the life cycle.
- Appendix F contains a Glossary of terms used in Dam Safety.

Table 1-3 shows how the contents of Volumes 1 (Chapters 2, 3, 4 and 5) and Volume 2 (Appendix A, B, C, D, E and F) of the present Manual are related and shares through examples, illustrations, Case Studies, the information that in some stage Dam safety Engineers will need in this assessment of the hydraulic Safety of the Dam .

Appendix	Identification	Chapter 2 Dam and Its Reservoir	Chapter 3 Spillways	Chapter 4 Outlets	Chapter 5 Energy Dissipators
<b>Appendix A: Failure Mode Identification</b>					
A1-FM1	Concrete dam overtopping				
A1-FM2	Embankment dam overtopping				
A1-FM3	Dam overtopping caused by gate failure				
A1-FM4	Spillway blockage by debris & gates jamming				
A1-FM5	Chute wall overtopping in ungated spillway				
A1-FM6	Cavitation damage in chute & energy dissipator				
A1-FM7	Hydraulic jacking in chute or energy dissipators				
A1-FM8	Erosion of chute foundation				
A1-FM9	Cavitation damage d/s of an aeration device				
A1-FM10	Malfunction of spillway flip bucket				
A1-FM11	Malfunction of stilling basin				
A1-FM12	Unstable hydraulic jump in the stilling basin				
A1-FM13	Ungated spillway toe erosion due to weir flow				
A1-FM14	Plunge pool erosion				
A1-FM15	Bank erosion due to poor flow exit				
A1-FM16	Bottom outlet blockage				
A1-FM17	Intake structure blockage				
A1-FM18	Erosion caused by inadequate Energy Dissipator				
A1-FM19	Piping in the embankment due to filtration				
A1-FM20	Piping in embankment dam due to flow				
A1-FM21	Piping in embankment due to structural failure				
A1-FM22	Gates and/or valves jamming				
A1-FM23	Malfunction of sluice gate operation				
<b>Appendix B: Case Studies</b>					
Guri Project	Energy Dissipation and Aeration				
Macagua Project	Energy Dissipation, Cavitation, Aeration				
Caruachi Project	Energy Dissipation				
Tocoma Project (UC)	Energy Dissipation				
El Guapo	Chute and dissipator erosion and dam failure				
Tauk Saum Project	Dam Failure by Overtopping				
Spencer Project	Dam Failure caused by ice-jam				
Maneri Project	Repair Spillway/Dissipater damaged by boulders				
Narayanpur Project	Spillway/Radial Gate				
<b>Appendix C: Retention by Means of Floating Barriers</b>					
-	Debris retention by means of Floating barriers (Tuff Booms)				
<b>Appendix D: Hydraulic Modeling-A compendium</b>					
-	Hydraulic Modeling-A compendium				
<b>Appendix E: Operational Safety of Hydromechanical Equipment</b>					
-	Operational Safety of Hydromechanical Equipment				
<b>Appendix F: Glossary of terms for Dam Safety</b>					
-	Glossary of terms for Dam Safety				

Table 1-3 Manual for Assessing the Hydraulic Safety of Existing Dams- Vinculation of Volume 1 and 2

### 1.5 Limitations

This Manual does not address the following aspects of the dam-reservoir system:

- Risk analysis except for Appendix A, where 23 cases of Identification of Failure Modes are presented. These Failure Modes are originated by Hydraulic causes or on hydraulic structures.

- Hydraulic design.

The main objective of the Manual is Hydraulic Safety and related Rehabilitation works. Some information on hydraulic design particularly in Non-conventional spillways is included.

- Supervision, Operation and Maintenance.

For the purposes of this manual, the boundary between O&M activities

and Rehabilitation is contained in the CWC publication: "Guidelines for preparing operation manuals for dams" (January 2018)

- Aspects related specifically to structural, geotechnics, and environmental concerns.
- Dams other than large dams as specified by "The Dam Safety Bill (2019)".
- Aspects of vandalism, sabotage and dam security.

## 1.6 Map of the Manual

Figures 1-5 and 1-6 shows how the user can be oriented according to their needs when reviewing the Manual, for Spillways and Outlets, respectively. It is basically a diagram of location of component along the structure and the related Chapter of component vs Hazard- Response –Consequence .

- Figure 1-5: Spillways - Various hazards/defects and their adverse responses/effects on hydraulic safety (Related chapters of this Manual).
- Figure 1-6: Outlet works - Various hazards/defects and their adverse response/effects on hydraulic safety (Related chapters of this Manual).

## 1.7 Relation with other CWC Guidelines

Form its mission statement, Central Water Commission promotes integrated and sustainable development and management of India's by using state of art technology and competency and by coordinating the stakes holder water resources.

Following its mission CWC have prepared through DRIP Projects 16 Guidelines and Manuals , covering different aspects of Dam Safety for capacity building in the process of management the Dam Safety issues of In the large portfolio of Indian dams.

The present manual for Assessing the Hydraulic Safety of Existing Dam, conforms part of this documentation prepared under DRIP project, and it is integrated to the broad concept of Risk Assessment and Management, which provides a global framework where all aspects related to dam safety are integrated to assist decision making.

For this reason, the use of all Guidelines and manuals developed for CWC under DRIP Project, should be integrated for they are directly or indirectly related to the Risk Assessment and Management process. Figure 1-4 shows how the present "Manual for Assessing the Hydraulic Safety of Existing Dams interact with other Guidelines and Manuals.

For instances, Hydraulic safety assessment is based on knowledge of the physical condition, operation and performance of the works and equipment obtained from both, available information and on-site inspections. This phase is fed with the interpretation of the failure modes (FM) which encompasses definition of hazards or loads (hydrological, hydraulic and anthropic) and analysis of the response of each component according to its characteristics and current state.

Failure modes are a basic tool for establishing a particular sequence of events that can result in improper operation, loss of reservoir purpose, disruption or failure of the dam or any of its appurtenances works.

Appendix A, Volume 2 of the manual, gathers a catalogue of a series of Failure Modes, may serve as a guide to complete the practical use of the manual, by including this tool of risk management and safety assessment through the process of Failure Mode Identification.

## **1.8 Hydraulic Safety Issues in the Present Manual and in other CWC Guidelines and Manuals**

Tables 1-4 to 1-7, is a reference for the readers where they can identify one topic of interest and see the guideline and/or Manual where he should investigate .

For instances, two topics, “Flood routing through the reservoir” and “Calculation of freeboard of the dam due to wave action”, which although eminently hydrological-hydraulic in nature, were previously included in the “Manual of Assessment of Structural Safety of Dams” given their direct relationship with the dam's crest elevation (Dam top elevation) requirement and have therefore not been discussed in this Manual

## **1.9 Publication and Contact Information**

This document is available on the CWC website <http://www.cwc.gov.in> and the Dam Rehabilitation and Improvement Project (DRIP) website [http //www.damsafety.in](http://www.damsafety.in).

For any further information contact:

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Central Dam Safety Organization,

Central Water Commission,  
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R. K. Puram, New Delhi – 110066,  
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## **1.10 Acknowledgments**

In preparing this Manual, works of various organizations in Australia, Canada, India, Spain, Switzerland , the United States, Venezuela and elsewhere have been drawn liberally.

Grateful appreciation is extended to the following organizations whose publications and websites have been sources of valuable information for this Manual:

- Australian National Committee on Large Dams.
- Bureau of Indian Standards.
- Canadian Dam Association.
- Corpoelec.
- CVG EDELCA.
- Federal Emergency Management Agency, U.S. Department of Homeland Security.
- International Commission on Large Dams (ICOLD).
- SPANCOLD.
- Swiss Committee on Dams.
- U.S. Bureau of Reclamation.
- U.S. Army Corps of Engineers

## RELATIONSHIP TO OTHER GUIDELINES AND POLICIES



Figure 1-4: Relation between Manual for assessing Hydraulic Safety of Existing Dams and other DRIP Guidelines and Manuals within the Risk Assessment and Management Process and Regulatory Framework

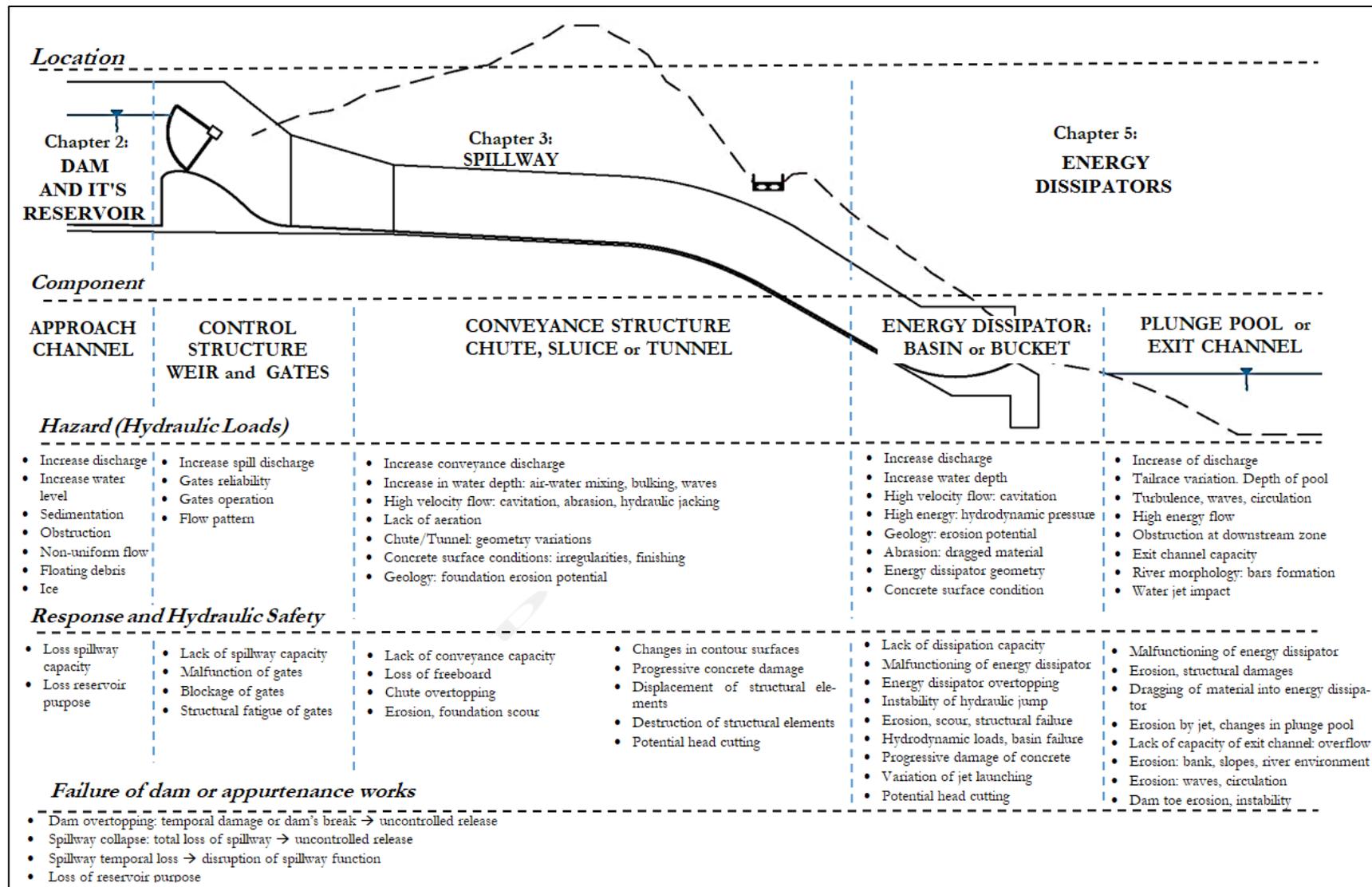


Figure 1-5: Various hazards/defects and their adverse responses/effects on hydraulic safety (Related chapters of this manual)

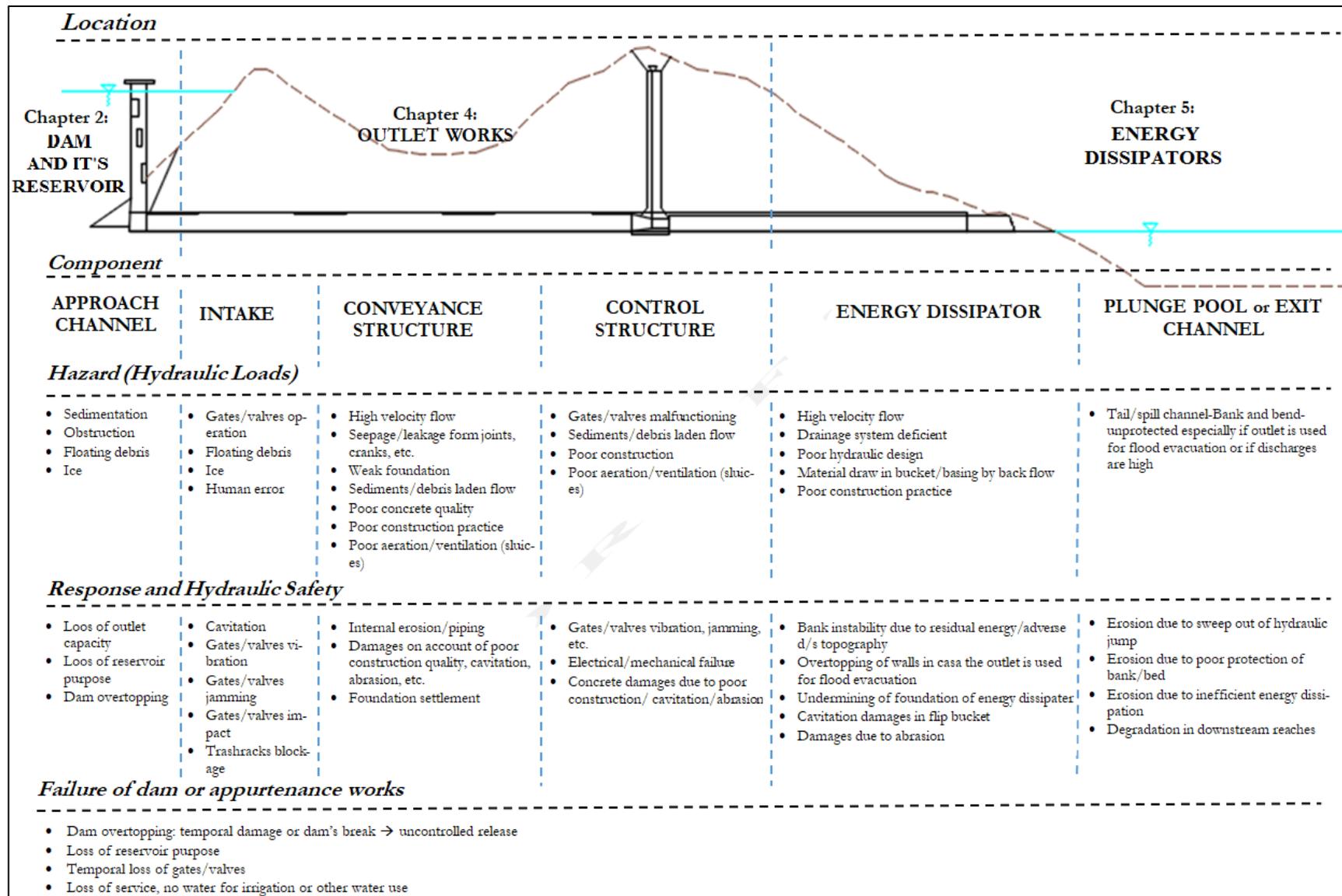


Figure 1-6: Various hazards/defects and their adverse response/effects on hydraulic safety (Related chapters of this manual)

Hazard/Adverse Response		Causes	Related DRIP Guidelines/Manuals
I	Reservoir Sedimentation	1. Reservoir Sediment Inflow	Handbook for Assessing and Managing Reservoir Sedimentation
		2. Bank Sliding	
		3. Wind Induced Waves	Manual for Assessing Structural Safety of Existing Dams
		4. Wind Run Up	
		5. Reservoir Operation	Guidelines for Preparing Operation and Maintenance Manual for Dams
		6. Rim Erosion	Out of scope of DRIP guidelines/manuals
		7. Reservoir rapid drawdown	
		8. Delta Formation	
		9. Bottom Deposits	
II	Dam Overtopping	1. Wave Run Up	Manual for Assessing Structural Safety of Existing Dams
		2. Reservoir Operation	Guidelines for Preparing Operation and Maintenance Manual for Dams
		3. Accommodation of Inflow Flood	Guidelines for Selecting, Estimating and Accommodating Inflow Design Floods for Dams
		4. Blockage due to floating debris	<a href="#">Manual for Assessing Hydraulic Safety of Existing Dams chapter 2</a>
		5. Blockage due to ice	<a href="#">Manual for Assessing Hydraulic Safety of Existing Dams chapter 2, Appendix A</a>
		6. Insufficient spillway capacity	<a href="#">Manual for Assessing Hydraulic Safety of Existing Dams Chapter 2, 3, Appendix A</a>

Table 1-4: Hazards/Adverse response/Causes connected with Dam Reservoir and Approach Channel-Related CWC Guideline, Handbooks and Manuals

	Hazard/Adverse Response	Causes	Related DRIP Guidelines/Manuals
I	Overtopping of dam	1. Blockage due to floating debris	Manual for Assessing Hydraulic Safety of Existing Dams -Chapter -2, Appendix C
		2. Blockage due to ice	Manual for Assessing Hydraulic Safety of Existing Dams -Chapter -2, Appendix-A, Appendix B
II	Gate / Valves Malfunctioning	1. Gate Vibration (in spillways)	Manual for Assessing Hydraulic Safety of Existing Dams -Chapter -2, Appendix-A, Appendix E
		2. Gate Vibration (in sluice)	Manual for Assessing Hydraulic Safety of Existing Dams -Chapter -4, Appendix E
		3. Gate / Valve Jamming by Ice	Manual for Assessing Hydraulic Safety of Existing Dams -Chapter -3, 4, Appendix C, Appendix B, Appendix E
		4. Gate / Valve Impact by floating debris	Manual for Assessing Hydraulic Safety of Existing Dams -Chapter -3, 4, Appendix C
		5. Gate / Valve operation by human error	Manual for Assessing Hydraulic Safety of Existing Dams -Chapter -3, 4, Appendix A
III	Concrete Damage	1. Cavitation	Manual for Assessing Hydraulic Safety of Existing Dams -Chapter -3, 4, Appendix A, B, E
		2. Abrasion	Manual for Assessing Hydraulic Safety of Existing Dams -Chapter -3, 4, Appendix – A, Appendix A, B,

Table 1-5: Hazards/Adverse response/Causes connected with Dam Reservoir /Approach Channel / Control section / Gate - Related CWC Guideline, Handbooks and Manuals

Hazard/Adverse Response		Causes	Related DRIP Guidelines/Manuals
I	Wall Overtopping	1. Increase in discharge	<a href="#">Manual for Assessing Hydraulic Safety of Existing Dams, Chapter 3, Appendix A,B</a>
		2. Flow bulking	
		3. Perturbed free surface	
II	Chute Cavitation	1. Poor concrete quality	Guidelines for Safety Inspection of Dams
		2. Flow velocity	<a href="#">Manual for Assessing Hydraulic Safety of Existing Dams, Chapters 3,4, Appendix A, B, D</a>
		3. Low boundaries pressures	
		4. Pressures pulsation	
		5. Poor ventilation (Sluice)	
III	Chute abrasion	1. Sediment / Debris laden flow	Handbook for Assessing and Managing Reservoir Sedimentation
		2. Side rock falling	<a href="#">Manual for Assessing Hydraulic Safety of Existing Dams, Chapters 3,4, Appendix-A</a>
IV	Concrete Chute Jacking	1. Poor construction practice	Guidelines for Safety Inspection of Dams
		2. Drainage system deficient	
		3. High velocity flow	<a href="#">Manual for Assessing Hydraulic Safety of Existing Dams, Chapters 3,4, Appendix-A</a>

Table 1-6: Hazards/Adverse response/Causes connected with Conveyance structure - Related CWC Guideline, Handbooks and Manuals

Hazards/adverse responses		Causes	Related DRIP Guidelines/Manuals
I	Undermining of Energy Dissipater Foundation	1. Poor protection at spillway toe	Guidelines for Safety Inspection of Dams
		2. Bank instability	
		3. Asymmetrical flows	<a href="#">Manual for Assessing Hydraulic Safety of Existing Dams, Chapter 5, Appendix A, 2</a>
		4. Flow circulation	
		5. High velocity flows	
		6. Poor geology	Guidelines for Safety Inspection of Dams
		7. Poor construction practices	
		8. Low tail water	<a href="#">Manual for Assessing Hydraulic Safety of Existing Dams, Chapter 5, Appendix A, B</a>
		9. Flip bucket design	
		10. Inefficient energy dissipation	
		11. Poor construction practice	
II	Cavitation	1. Poor project layout	Guidelines for Safety Inspection of Dams
		2. High velocity flow	<a href="#">Manual for Assessing Hydraulic Safety of Existing Dams, Chapter 5, Appendix A, B</a>
		3. Pulsating pressures	
		4. Flow unitability	
		5. Macro turbulence	
		6. Due to return flows	
III	Abrasion	1. Material in basin	Guidelines for Safety Inspection of Dams
		2. Sediment laden flow	<a href="#">Manual for Assessing Hydraulic Safety of Existing Dams, Chapter 5, Appendix A, B</a>
IV	Overtopping	1. Increase in discharge	<a href="#">Manual for Assessing Hydraulic Safety of Existing Dams, Chapter 5, Appendix A, B</a>
		2. Flow turbulence / waves	
V	Concrete Jacking (Stilling Basin)	1. Poor construction practices	Guidelines for Safety Inspection of Dams
		2. Sweep out of hydraulic jump	
VI	Tailrace Channel -Bank and Bed Erosion	1. Poor protection of bank/bed	<a href="#">Manual for Assessing Hydraulic Safety of Existing Dams, Chapter 5, Appendix A, B</a>
		2. Energy dissipation inefficient	
		3. Degradation in downstream reaches	
		4. Flow circulation	

Table 1-7: Hazards/Adverse response/Causes connected with the Energy Dissipator -Related CWC Guideline, Handbooks and Manuals

## Chapter 2. DAM AND ITS RESERVOIR

### 2.1 Overview

Reservoir is the volume of water impounded by construction of a dam in order to use it for the purposes for which the project has been designed.

The reservoir volume guarantees the supply of water or energy during the months of low flows (dry season). Its dimensions determines the volume available for storing the flows during the rainy and thawing seasons. In India most of runoff comes during the monsoon months and is stored in the reservoir for use in non-monsoon months.

### 2.2 Lessons

Numerous incidents in reservoirs have compromised the integrity of the dams, affected their operation and, in some extreme cases, caused the dam to fail. Below is a succinct account of some emblematic cases where the incidents generated by the

reservoir has caused a partial or total failure of the dam.

The need to extend dam safety investigation to include the reservoir rim was dramatically illustrated by the 1963 landslide on the valley wall of the Vajont Reservoir in Italy (Kiersch 1964). In that case the wave caused by the slide of rock (some 200 million cubic meters) into the reservoir, overtopped the dam by about 125 meters. Although the dam withstood the impact with only minor damage, the wave continued downstream into the town of Longatone killing an estimated of 2,000 people. This aspect requires a thorough Geological assessment of the reservoir rim and banks, which is however beyond the scope of this manual.

Reservoir outflow can also be severely reduced by obstructions of outlets, spillways and intakes, impacting the spillway capacity and the hydraulic safety of the dam.



Figure 2-1: Vajont Dam and the left bank Sliding.

The dam did not fail but the reservoir was filled with material from the slide (Kiersch, 1964)

The obstructions can be caused by slides on account of adverse geological features in the abutments/reservoir rim, debris, siltation, landslides or, most common combination of these factors (See Figures 2-1 and 2-2).

If the intake is located without taking into account the dead storage capacity, the intake structure may be in danger of getting obstructed by a mixture of trash, sediment and debris. What follows is a loss of withdrawal capacity due to a loss of intake due to obstruction.

The most prominent threat for a reservoir is the accelerated rate of loss of storage due to reservoir sedimentation. This is probably the most recurrent problem in many of the

reservoirs worldwide. This issue is treated with more detail in the *“Handbook for Assessing and Managing Reservoir Sedimentation”* (CWC, 2018).

Obstruction of hydraulic outlets/intakes can also be caused by ice. Ice formation, limits and even prevents winter normal operation of the hydraulic structures. Presence of ice at the entrance of power plants, spillways, intakes, outlets and other hydraulic structures, may create operational problems, leading to subsequent damage to the infrastructure. Spencer dam failure in USA is one example caused by this phenomenon (See Figures 2-3 and 2-4).



Figure 2-2: Floating debris obstructing the Palagnedra Dam spillway during the 1978 flood (Swiss Committee of Large Dams, 2017).



Figure 2-3: Spencer dam in normal operation



Figure 2-4: Spencer dam failure  
Tainter gates were not able to open because some were frozen shut.

A loss of dam freeboard can be induced by wave action, run up, wind gusts, during severe storms or by erratic reservoir operation during the passing of ordinary or extraordinary floods that may require additional structural/non-structural measures to accommodate the Inflow Design Flood. This important issue is discussed in various chapters of this Manual. It is covered with more detail in the “Guidelines for Selecting and Accommodating the Inflow Design Flood for Dams”(CWC, under preparation), and in the “Manual for Assessing the Structural Safety of Existing Dams”, (CWC, 2020), both written as part of DRIP Project.

Not only the reservoir rim is of a need for risk assessment, but also the connections between the spillway and outlet works ,with the reservoir. This brings up the approach channel or the preference water way used by the flow to enter an outlet work, intake or spillway. Approach channel design should enhance flow performance by good flow connectivity between the spillway and outlets with the reservoir.

For small reservoirs, rapid drawdown may be converted in a hazardous operation condition a hazardous operation condition,

for embankments dams, which are not properly design for this type of operation. This aspect is covered in the “Manual for Assessing the Structural Safety of Existing Dams”, (CWC, 2020) written as part of DRIP Project.

**REFER TO APPENDIX B-4**  
There you can learn from the failure of Spencer Dam, as a consequence of a Ice Jam

### 2.3 Hydraulic Safety Assessment

Man-made reservoirs are generally developed to provide flood risk management, irrigation, water supply, power-generation and various other benefits like recreation.

However, there are many ways the reservoirs can affect the safety of the dam that is impounding the reservoir. The safety review of a dam/reservoir project should cover seismic, hydrologic (potential over-topping during an extreme flood event), and seepage issues in the dam. The safety of the dam, the spillway and related structures can be impacted by the state of the reservoir in many different ways.

A quick review of defects/hazards which can arise from the reservoirs and approach channel of the hydraulic structure being those intakes, spillways and outlet works, is described in Table 2.1.

All types of defects or hazards that may be present in the reservoir falls into three types of main effects:.

1. Effects that result in loss of reservoir storage by sediment driven by the river.
2. Effects that result in loss of reservoir storage by massive landslides
3. Effects that make the reservoir loose freeboard, such as waves, wave run up, etc.
4. Effects that make the spillway/ outlet/ intakes to loose discharge capacity, due to blockage by debris of diverse type.

All the above cited effects create without exception, a loss of reservoir capacity to safely pass floods. .

When trash racks are used, their proper design and placement along with regularly scheduled maintenance and cleaning of debris from the racks can prevent such incidents. The design of racks must consider such factors as the intended use of the reservoir (hydro-power, water supply, irrigation, flood control, etc.), types of gates, and maintenance requirements (US Bureau of Reclamation, 1974). Where log booms are used to prevent obstructions to spillways and intakes, accumulated debris should be continuously removed and inspection made for damaged, corroded, or inadequate log booms.

	HAZARDS / DEFECTS	CAUSES
1	Reservoir Instability/ Bank Sliding	Geology/seismic/ Reservoir Operation
2	Rim Erosion	Waves, Reservoir Operation /Drawdown,
3	Reservoir Sedimentation	Sediment production in the river basin
6	Bank Storage	Banks Geology
7	Reservoir Induced Seismicity	Seismic/geological
8	Back water flooding	Back water due to terrain landmarks
9	Ice	Low temperatures
10	Wind induced waves/Wave run up	Wind
11	Reservoir Operation	Policies/manmade errors
12	Rapid Reservoir Drawdown	Policies/ manmade errors
13	Reservoir Sedimentation/Delta formation	Sediment production in river basin
14	Reservoir Sedimentation/Bottom Deposits	Sediment production in river basin
15	Floating Debris	Debris production in the River Basin
16	Inflow Flood accommodation in the reservoir	Poor estimation of Freeboard / Reservoir Operation
17	Slide Bank Erosion	Bank instability/Seismic effect

Table 2-1: Dam and its Reservoir related hazards

### 2.3.1 Floating Debris

#### Introduction

Floating Debris is a problem at many hydraulic structures including hydroelectric power plants and spillways. Debris represents a potential operational as well as a dam safety hazard. Debris can clog the screens/trash racks in power intakes, block spillway openings, reduce reservoir flood storage as well as exert additional load on the dam structure when accumulated in large quantity near the bottom of the dam over time. Accumulation of floating debris in the reservoirs can have significant negative impacts in operations and functions of a dam and may lead to dam safety problems.

Floating debris such as large wood and other anthropogenic waste materials are often carried during floods, which can lead to blockages of dam spillways. In particular, obstruction of dam crests or gates may considerably reduce the discharge capacity of the spillway and cause unacceptably high-water levels in storage reservoirs. The required freeboard clearance can then no longer be guaranteed (See Figure 2.5).

In addition to assessing the risk of obstruction, the fundamental question is whether large wood should be retained or passed through. Both require a

corresponding design of a dam the spillway, or appropriate measures in the reservoir .

In recent years much driftwood has combined with debris and sediments, due to heavy downpours over mountainous rivers, to result in various operational problems and loss in disastrous damage and loss of life in the lower reaches of the river. Recent attention has therefore been focused on driftwood countermeasures aimed at driftwood transported by debris/sediment water flows. There are no generally accepted and valid guidelines for dealing with large wood and floating debris at dam spillways until now (Swiss Committee on Large Dams, 2017) .

To address the issues related to floating debris on dam safety, three areas of discussion are relevant , namely:

- Potential impact of floating debris on dam safety
- Quantification of floating debris
- Mitigation methods to reduce the potential impact of floating debris on dams



Figure 2-5: Floating debris in front of Thurnberg Dam Spillway, Austria, during the extreme flood of 2002

### **Mechanism of producing debris**

Major mechanisms by which debris is introduced in the rivers/watercourses include wind, storms and wave action. The erosion of the river/reservoir banks causes trees to topple into the water. Wind, storms and wave action can also carry the debris from their natural sites of storage, and other lighter material to the water bodies. Other causes of producing debris are ice break up and the action of ice storms due to which trees can break and fall by weight into the watercourses. Forest processes and practices, debris jams, and manmade trash are some other factors (Swiss Committee on Large Dams, 2017) .

### **Characterization of Floating Debris**

Floating debris comes in many forms, including, but not limited to the following:

- Natural tree trunks and roots (dead or fresh wood)
- Timber from logging or deforestation
- Vegetative debris left over from timber harvest and logging operations
- Vegetative masses, such as reeds, bushes, and other aquatic plants and materials in the rivers upstream of the dam
- Domestic wastes, such as plastic bottles and trash deposited by people in the watershed above the reservoir; houses, even boats and abandoned wrecked cars and many other types of debris, which could get washed down from rivers upstream of the reservoir during a major flood event.

### **Impact of floating Debris on dams**

Accumulation of floating debris in the reservoirs can have significant negative impacts in the operations and functions of a dam. Main impacts of floating debris fall under the following categories. Impact on rigid structures, clogging at trash racks at

intakes, and , loss or reduction of spillway discharge capacity.

Debris impacts on a reservoir and dam appurtenances is a topic where little background research or information has been compiled.

However, literature available in US Federal Highway Administration (FHWA), US Army Corps of Engineers (USACE) publication, United States Society on Dams (USSD) and ICOLD conferences/ publications can be referred to.

### **Impacts of floating Debris on Spillways, Intakes and Outlets (on Hydraulic Structures)**

A number of impacts are caused by floating debris in hydraulic structures and hydro-mechanical equipment's. On occasions, dam gates can get stuck, while partly open by debris intrusion. Floating debris can damage the upstream slope of dams through wave action, which hammers debris against the dam walls and other structures. Large tree trunks may impact on trash racks and power plants and spillways gates.

**REFER TO APPENDIX A**

**For Failures Modes involving Debris.  
See FM-3, FM-4 And FM-8**

Further in general, most of the spillways are not specifically designed for debris passing / clogging. Present practice for design of intakes prove inadequate for it does not incorporate debris loadings event that are critical for reliable and safe operations of the intake. Clogging of a low outlet intake have been causes of dam failures in many dams (Wark, 2015) .Debris clogging is a frequent hazard for low level intakes. some of the spillways especially in mountainous region. At the time of this literature review, no dam safety regulations, in the United States or other parts of the world, regarding debris passage through spillways were found. Also, no guidelines for the evaluation of malfunctioning spillways (gated and ungated)

given the presence of floating debris transported by flood flow were found.

Significant cases of spillway clogging due to floating debris are described (Wark et alia, 2015. Wallertstein et alia, 1997) includes the following lessons learned (Table 2-2).

#	LESSON LEARNED
1	Debris and floating log, are capable of obstructing spillways, leading to dam overtopping during a flood scenario.
2	Some measures can be taken to prevent overtopping, for example, keeping a large bay width between spillway piers, removal of spillway bridge to provide a free passage for debris and logs to flow down the spillway along with flood waters.
3	Setting spillway piers about 12m apart, in order to keep reasonable free width for passing of flow with debris in a flood situation
4	Analysis of vegetative typologies of forestry practices in the watershed to understand and assess potential of the river watershed to produce debris loading
5	Pier spacing of the spillway should be at least 80 % of the maximum size of trees moved by the water current
6	Closed conduits are more vulnerable to clogging than open conduits
7	In free surface spillways, avoid flow contractions, sharp bends and rough walls
8	Drum, sector and flap gate should be preferably used to avoid gate clogging
9	Lift gates should be avoided unless there are a large number of openings due to the danger of trees being drawn below their lower edge during closure
10	Considering large size of water passages, for provision for large tree and other debris
11	It is usually impossible to remove all floating debris during a flood event because of the large volumes. In addition, if the dam spillway is activated, a partial discharge of floating debris via the dam spillway can hardly be prevented.
12	Increase hydraulic capacity of tunnels spillway to 5,000-year design flood with a minimum 5 m tunnel diameter.
13	Use open conduits when possible. If there are closed conduits, use smooth walls, no contractions or obstructions and no sharp bends
14	Concentrate the intake structure in one opening and make the invert of the intake as low as possible
15	Use radial gates in spillways; avoid vertical lift gates.
16	Avoid trash-racks at spillways as they compromise the design flood capacity
17	Try to intercept debris upstream of a reservoir, e.g. debris basins, debris retention posts
18	Use physical models in the design of spillways with high exposure potential to large amounts of floating debris

Table 2-2: Lessons learned – Spillway clogging due to floating debris.

**Concepts for dealing with Floating Debris in dams**

In general, three (3) options are available with dealing with Floating Debris in a Dam-Reservoir system:

- 1) Measures taken in the catchment area to minimize log accumulation;

- 2) Retaining debris and removing it from the reservoir.
- 3) Allowing debris to pass through the dam spillway.

**Floating Debris Assessment / Quantification / Flood Floating Debris events**

- Prediction of the Floating Debris Volume

Different methods are available to predict the volume of Floating Debris that may reach a given reservoir during a flood based on catchment area, past flood events or floating debris observations. Methods gives very different results - so a sensitivity analysis is recommended before arriving at a definite figure. (Wark, 2015).

The evaluation of the drainage basin area is an important factor to define the approximate floating debris volume. US Corps of Engineers study is based on multiple linear regressions, hydrologic and meteorological. In river basin that shows significant floating debris with each flood, a correlation of lost forestry and/or amount of floating debris in the reservoir, can give a preliminary idea of the debris production of the basin. Methods such as GIS, Lidar, Analysis of aerial photos before and after the flood, may give good insight into the volume of floating debris.

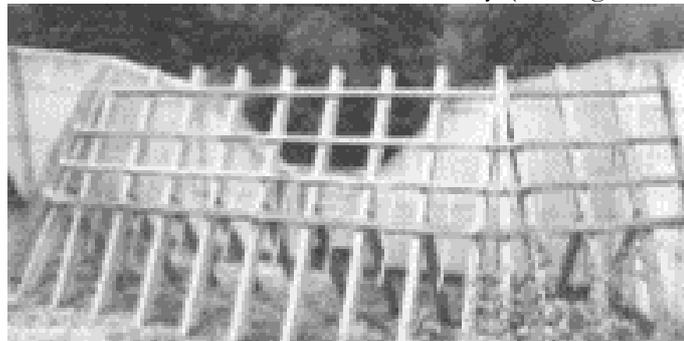
As a conclusion, the volume of transported floating debris is important for hazard evaluation but difficult to determine in the field. Although several empirical methods exist to predict the floating debris potential of a given drainage basin, comparisons with field data indicate a large scatter. A detailed study of the drainage basin is therefore necessary to produce more reliable results.

- Countermeasures - Mitigation Impacts of Floating Debris

Mitigation methods to reduce the potential impacts of floating debris on dam safety can be classified into three broad areas: watershed management, design and operation considerations, and capture of floating debris upstream of dams. A number of different mitigating methods are also discussed in the Canada Dam Association Dam Safety Guidelines (1999)

### Watershed Management

Preventive protective measures in the river basin are always of great help to reduce the volume of debris by particularly avoiding landslides and bank erosion (Swiss). Stable banks and river flow slopes, stable well anchored trees should be aimed for. Logs, not well anchored trees should be eliminated. Some exposed trees near the river line should be managed according to the environmental rules. Trees management including tree cutting and/or eliminating should not result in conflict with environmental issues. Management of river basin areas and banks for the purpose of reducing potential volume of floating debris is difficult and costly. Stakes holders such as dam operators, forestry services, conservation office need to have a agreement for the above requirement.. Large detention structures or basin can be designed and planned in the river catchment, nearby the river/reservoir line, in depressions or sites where wood can be jammed and collected before entering the water body (See Figure 2-6).



a) By trash-rack



b) By cable net



c) By barrier of cable with large wood elements

Figure 2-6: Large wood retention

Other good practices to be exercised in the river basin include (Wark, 2015):

- Constructing debris dams upstream of reservoir, and periodic removal of accumulated debris
- Coordinating and minimizing strip clearing of the forests
- Providing adequate drainage of slopes to prevent soil erosion and potential landslides which carry vegetation to streams leading to the reservoir.
- Rapid re-planting and land treatment in logged and mined areas to prevent excessive erosion
- Creating timber barriers downstream of logged areas to prevent movement of downed timber downstream
- Providing a no-construction buffer zone along streams and the reservoir proper
- Constructing temporary storing yards for harvested logs and any buildings outside of the 100-year floodplain at a minimum, if the watersheds are being logged for timber or are inhabited
- Working with the forestry companies and/or respective jurisdictional governments to implement forest conservation measures and zoning plans to reduce the potential of human-generated debris on the watersheds

Because clearing of lands for the reservoir area is one of the integral construction

activities for a hydro-project, good planning and execution are also essential relative to the reduction of potential floating debris problems for the project operations in the future. Many of the issues with floating debris could be prevented if there is a comprehensive and effective reservoir clearing program. In addition, much of the negotiation with landowners and forestry companies in the project watersheds to maintain good watershed management practices could also be formulated at the beginning of the project (Wark).

In general, a forestry specialist needs to conduct a study of characterization of forest species for commercial exploitation and for defining the potential volume to be converted into floating debris.

To minimize the generation of floating debris in a watershed which has been logged, it is important to have a detailed characterization of the forest debris in the reservoir area including;

- a) Unburned debris from cleared sites,
- b) Debris from old burned sites, and
- c) Debris left by the natural clearing agents, like ice, wind, or forest fires.

#### **In-stream Measures - Sabo Dams**

Known in Japan as Sabo Dams, the same have been built in many countries and they have performed well. They can be built with different designs depending of the material it is expected to trap (See Figures 2-7 to 2-9).

All these dams have to be provided with access roads and equipment to remove the trapped debris and for creating space to receive and trap debris in the next flood season.



Figure 2-7: A Steel Check Dam.



Figure 2-8: Open steel check dams for debris trapping.



Figure 2-9: Close up of Sabo dam

### Passing of Floating Debris

During the flood season, most of the debris flow is transported; passing downstream is the only option. The natural structure which will pass large amounts of debris flow is the spillway. However, debris should pass without blocking the structure. For this purpose, spillway bays should be designed with large dimensions and without obstructions to allow for passing the debris. Figure 2-10 shows an example of modifications made on spillway piers and

bridge at Palagnedra dam (Switzerland) to allow free passage of debris. Some measures which can be taken up on the upstream to trap and remove logs and debris have been described earlier in this chapter. Passing of the tress and debris by using the spillway structure alone shifts the problem to the downstream river reaches where debris will be routed through the river course, bridges sites, etc.

SEE APPENDIX A

To identify probable failure mechanisms of a dam with high presence of debris see failure modes FM-4, FM-16 and FM-17

### Design Considerations for Spillways and Reservoir Operation

Reservoirs generally allow for the removal of accumulated debris, if accessibility of large machinery is ensured. At low velocities the floating debris can be collected and removed by boats. This can prevent wooden logs from reaching the dam spillway in the event of a flood or sinking and obstructing and / or impairing the functioning of spillway, penstocks or low-level outlets.

Fresh wood usually remains buoyant for several months (Zollinger 1983), which means that removing such floating wood twice a year may be sufficient. Log wood is usually not distributed over the entire reservoir, but is blown by wind in bays or on certain shore areas. Floating barriers provided on reservoirs have proven themselves useful for collecting wood at the surface of reservoirs. However, forces are usually too strong during floods due to the high amount of wood & logs. Floating barriers are therefore not very reliable, and have also exacerbated problems when broken.

In case no measures are provided to trap/ remove floating debris upstream of the reservoir (especially in locations where there is a heavy debris load) ,then in the event of a flood carrying a large amount of floating debris, it may be almost impossible to

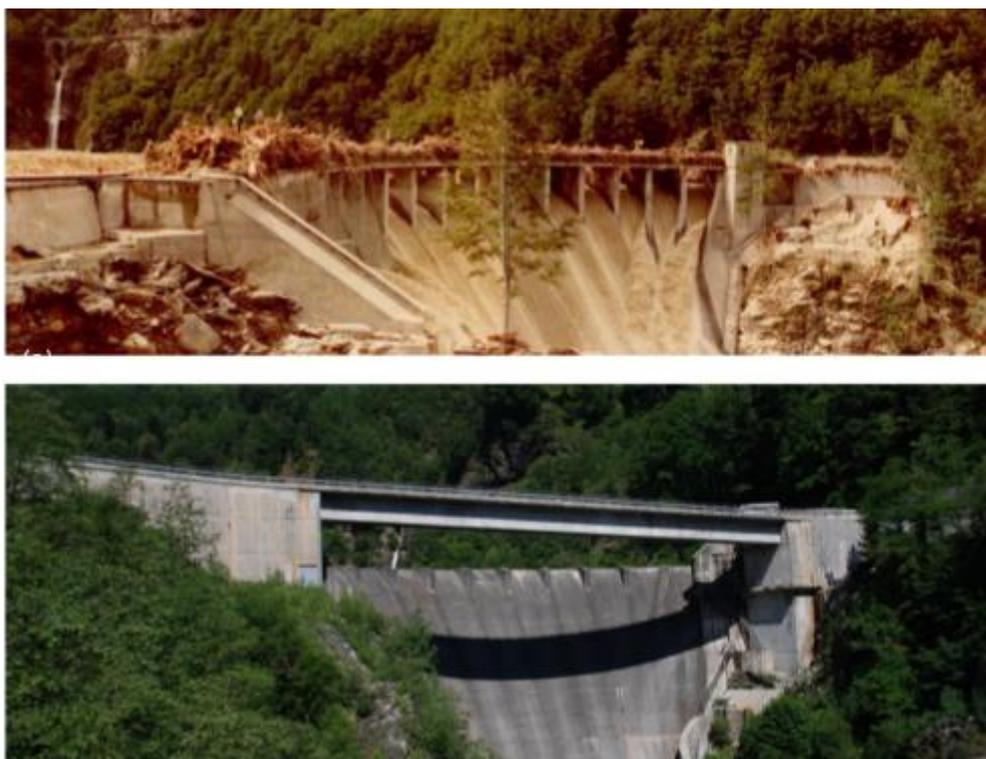


Figure 2-10: Palagnedra dam (Switzerland).

remove it all (trapped in front of floating barriers in the reservoir or in front of trash racks of power intakes) by excavators or rake cleaning machines (see Figure 2-11) due to the large amount of floating debris, flow velocities and the wedging of debris.



Figure 2-11: Clearing a blocked dam.

It is therefore advisable to keep debris from entering the dam spillway right from the start by structures in upstream reaches of the river, if it cannot be passed through the spillway without blocking. Floating debris that has been intercepted at upstream either in watershed structures, in stream structures, floating log barrier in the reservoir etc. can be removed from the reservoir after the flood,

with associated costs for removal, transportation, and disposal.

In areas very prone to floating debris, this feature should be incorporated in the design of the spillway, being a structure of paramount importance for the safety of the dam. The spillway design should guarantee passing of floating debris and to mitigate potential impacts of floating debris in dam safety.

Various measures that can be incorporated in the spillway design are provision of large size bays, avoiding a spillway bridge etc. to enhance floating debris passage.



Figure 2-12: Debris Visor (Swiss Committee on dams, 2017)

Some designs include two spillway sections, one gated for normal spillway operations and another one gated/ungated with a high crest level to pass the floating debris. This design has been adopted in many Hydro-Electric projects in India in the Himalayan region. In this case, floating debris/logs are passed from the gated/ungated spillway with a high crest level. Further it should be checked that the floating debris material in the downstream will not jam near the powerhouse where it may raise the tail water levels thus entailing a loss in energy generation.

Some special structures have been developed for those areas where there is a lot of wood debris; most of these structures are developed from physical model investigations

**SEE APPENDIX D**  
Where the utility of physical and mathematical models in the prediction of the behavior of hydraulic structures is analyzed

**Floating Debris Diversion Structure:** An example of this structure is shown in Figure 2-13. Located adjacent to the gated main spillway of the project, it consists of a fixed weir which streamlines the debris to the left bank of the service gated spillway. All features of the performance of this structure should be model tested.



Figure 2-13: Front view of floating debris diversion structure

**The Debris Visor:** Its area is larger than the spillway passage area. For this purpose, a curved or any other suitable shape can be considered to increase the spillway length (See Figures 2-12 and 2-14). However, it may be difficult and costly to build. Its behavior should be model tested.



Figure 2-14: Operation of the Visor during a flood.

### Structural measures for passing of large wood

#### Adjustment of spillway dimensions

- Ensure large dimensions of passage to account for floating debris.
- Remove/Avoid intermediate piers, spillway bridge and vertical lift gates.
- Flap, drum and radial gates are less susceptible to clogging by floating debris

#### Design of spillways inlets and transit structures

- Dam Spillways should be as smooth as possible, rounded and built without installations
- Avoid siphon spillways
- In gated spillways operate the gates to concentrate the flow towards the center, operate unsymmetrically, do not open two consecutive gates to avoid wedge debris formation.
- Use rounded piers.
- Bridges and pedestrian passes should have a clearance of 1.5-2 m above the maximum water level, they will preferably be removable or, can get

washed by the flow in case of an emergency

- Spillway bays should be at least 10- 15m wide.

### Design of Energy Dissipators structures

- Design should consider large wood logs, which travels at large velocity with great potential of damaging protruding structures such as baffles blocks, chute blocks, end sill etc. Increase width of stilling basin to account for log movement. Also the appurtenant blocks should be avoided where the debris load is high.
- Ascending apron stilling basin has proved beneficial for energy dissipater performance (see Figure 2-15).

**SEE APPENDIX D**

**Hydraulic Modeling for more details on physical models and micro models of hydraulic works**

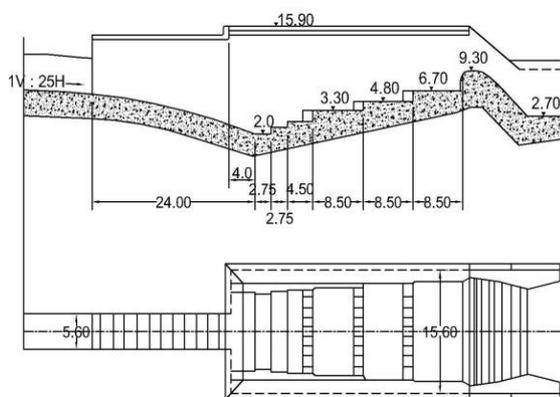


Figure 2-15: Ascending apron stilling basin

### Retention measures

#### Dam spillway protection with trash racks

- As a rule, the installation of trash-racks should be done in the front of the spillway (see Figure 2-16).



Figure 2-16: Spillway's Trash-racks.

- Racks can prevent blockage of movable parts thus guarantee movement of gates, flaps, etc.
- Flow at rack should be in the order of 1m/s.
- Rack can be completely blocked but area has to be sufficient to let flow pass to the control structure.

However, there is perhaps no such case history in India.

#### Retention by means of floating barriers (Tuff Booms)

- For low flow velocities they can work well and are useful tool for retaining and guiding wood logs (see Figure 2-17).
- For high velocity, flood situations with large volume of Log wood, robustness is not guaranteed, since many failures are reported.

**REFER APPENDIX C**

**For more information about floating barriers (Tuff booms): selection, sizing and design**

- Barriers should not be placed near the spillway rather at some sufficient distance upstream to prevent forces due to water currents.
- They can be equipped with underwater net to further reduce passing of wooden logs

- They serve to stop floating logs to pass and/or redirect debris to part of the lake where they are away from critical areas



Figure 2-17: Tuff Booms Barrier

Some authors find them useful only in limited cases, when logs are small or medium size. They find them not suitable for handling large logs.

- Aspects to consider in the design of a Tuff Boom barrier include chain stability and attachment to the shore, wear and tear of the temporal floating elements, change of wood buoyancy due to wood saturation, detention capacity of water by the wood log and, fluctuation of water levels in the reservoir
- Forces on the barrier are considerable
- Barriers should be removed in winter due to deterioration by ice, as applicable.
- Consider flexibility of the barrier in reservoir with water elevation changes

#### Operational measures

- To avoid obstructions, it is better to open few bays with large openings than to open many bays with small openings.

- In spillways with gates, flow should be concentrated in the center bays to avoid clogging at abutments
- Asymmetrical operation helps to avoid clogging (see Figure 2-18).



Figure 2-18: Obstruction in a three-bay system.

#### Hazard assessment diagram

When examining a dam and its potential vulnerability to floating debris the following diagram can be roughly followed which includes : Investigation of the spillway, flow conditions, dimensions, and to determine the impact in flood load cases, volume of large wood, dam spillway hydraulics, review the recommendations for minimum spillway dimensions, and estimate the blocking probabilities, assess the obstruction consequences and, decide whether there is a risk for the dam due to large floating material, develop the measures to reduce the risk of the dam as it is shown in Figure 2-19.

### 2.3.2 Ice and Frazil Ice

#### Introduction

Similar to the effects of floating debris, ice can generate obstructions and other alterations in the hydraulic structures of the dams that compromise their operation and, eventually, cause an incident in the intake or spillway structures. See Figure 2-4 (Spencer dam failure) for an example. From the same, it can be observed that how a dam failure is produced as a result of the blockage generated by the presence of ice (also see Figures 2-20 and 2-21).

Problems associated with the mechanical in-operation of the gates and valves due to low temperatures are not analyzed here; however, problems associated with the obstruction of intakes and spillway structures are discussed below. Blocks of ice transported by water currents cause erosion of the river banks/reservoir banks, the fall of trees from the banks of the river or the contour of the reservoir in the vicinity of the dam, obstruction of its relief or intake works as well as major impacts that may affect the structure.

Additionally, in cold regions and even below the surface of the water, suitable conditions of temperature variations cause the formation of ice crystals (frazil ice) that adhere to the trash-rack bars.

The problems derived from the clogging of power intake structures generate

inconvenience to the users of these systems in the ability to fulfill the commitments..

Special importance is to be attached to hydroelectric generation dams where clogging problems can prevent electricity supply in times of low temperatures when the demand for electricity is high and the population's dependence on the supply is greater. Efforts to mitigate the effects of ice on hydraulic structures are tackled in two ways:

- Prevent the accumulation of ice blocks in the vicinity of the spillway and outlet work.
- Avoid the formation of frazil ice in the trash-racks of the intake works.

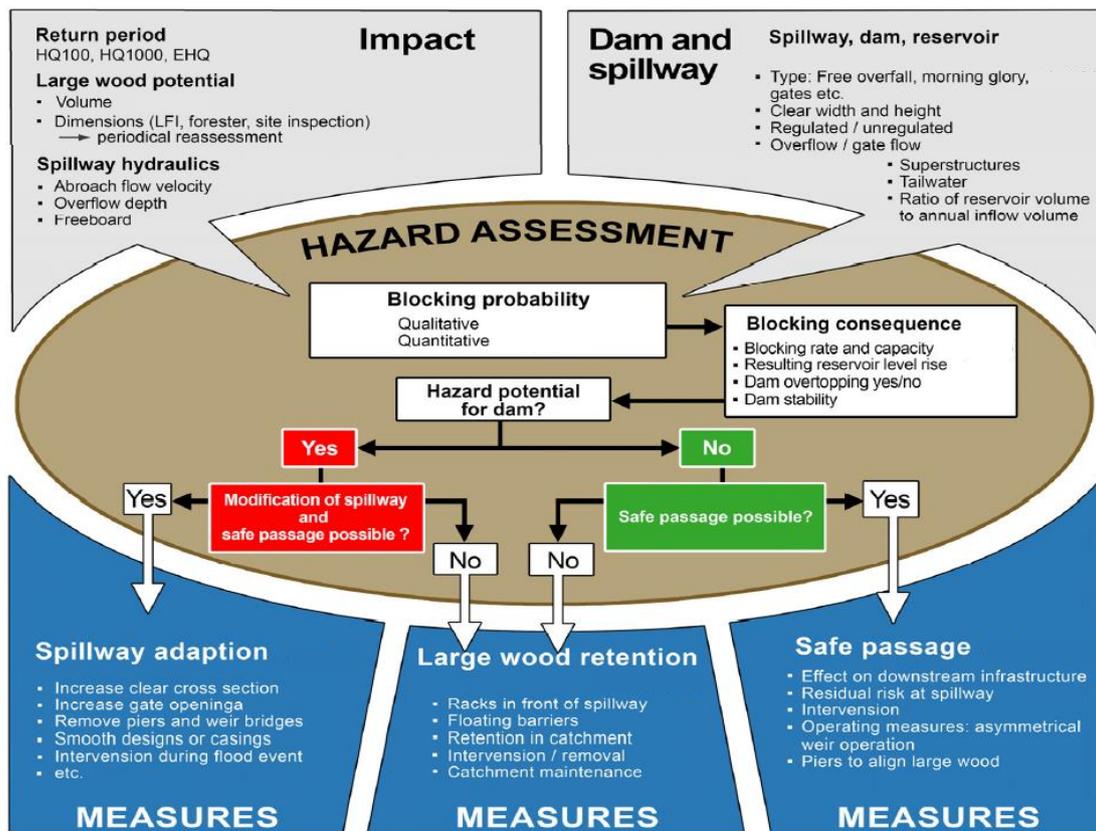


Figure 2-19: Hazard Assessment (Swiss Committee on Dams, 2017)

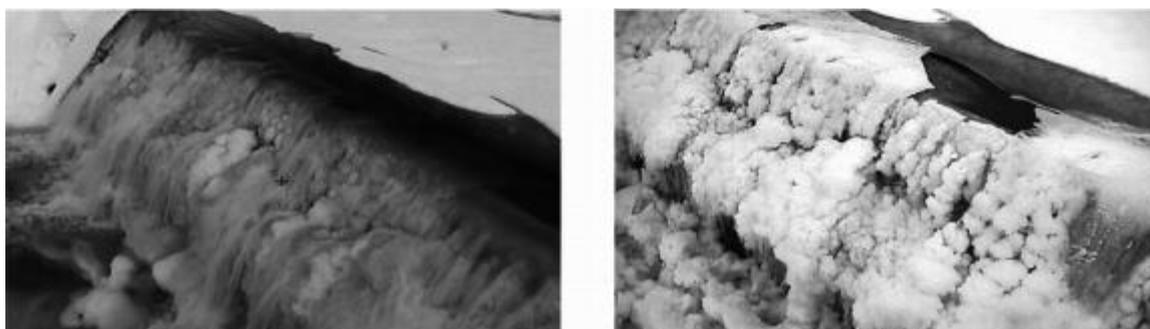


Figure 2-20: Ice cover on the Coanda-effect screen  
a) A complete frazil ice blockage of the screen b) Solid ice cover on the screen (Novik 2014)



Figure 2-21: Ice accumulation in spillway

### Effect of ice on dams

Accumulation of ice blocks in the vicinity of spillway and intake works can cause operational problems, degradation of concrete surfaces, structural overloads, blocking of mechanical elements and impacts of varying magnitude.

Those spillways that have closed ducts between their components are more sensitive to the presence of ice blocks. It is worth mentioning the Morning Glory Spillways, tunnels, and any other discharge carrying closed conduits are more susceptible to this problem.

Due to their buoyancy, the ice blocks can be removed from the most vulnerable areas of the dam with floating barriers that impede their progress with the flow of water inside the reservoir.

### 2.3.3 Freeboard

As mentioned in Chapter 1 of this Manual, one fundamental requirement for assessing the Hydraulic Dam Safety is the safe passing of the revised Inflow Design Flood (IDF). The revised MWL is determined by flood routing studies. The freeboard available above the revised MWL is required to be checked for its adequacy or otherwise.

#### Reservoir-Freeboard

Freeboard required in Embankment dams.

The freeboard is the vertical distance between the maximum water level in the reservoir, and the top of the dam. In general, most important components of the freeboard of a given dam consist of wind set up and wave run up.

Freeboard for embankment dams should be adequate enough to prevent any overtopping of the dam by either frequent or infrequent high waves that might interfere with efficient

operation of the project, or cause a dam breach and failure.

The availability of adequate freeboard reduces the risk and it can be considered as a safety factor against the uncertainty related to the flood. Freeboard associated risks are normally higher in embankment dams than in concrete dam, since the concrete dams can pass flows without experiencing much damage.

IS (Indian Standards) provides the basis for the definition and formulation of the Freeboard for Embankments dams. (IS 10635: 1993). Factors considered for estimating Freeboard include, wave characteristics particularly wave height, wave length, wind set up above the still water level, slope of the dam and roughness of the pitching. Freeboard does not account for effects of earthquakes, settlement of dam and dam foundations and earthquakes seiches.

Saville’s method has been used for computation of freeboard in IS 10635. The procedure to be followed for computation of freeboard is given in the above Indian Standard where an example of the computation has been

given for illustration. As per IS 10635 the normal freeboard (From FRL to dam top level) should not be less than 2m. and the minimum freeboard (From MWL to dam top level) should not be less than 1.5m for Embankment dams.

However, as the above procedure does not account for wind duration and gives conservative freeboard values, a more realistic procedure has been developed based on USBR guidelines on the subject and the book Advanced Dam Engineering by Robert B Jansen for existing dams and is included in “Manual for Assessing Structural safety of dams”, (CWC,2020).

**Minimum freeboard (Above MWL)**

A minimum freeboard is defined in the country guidelines of some countries such as Italy, Japan and Switzerland. See Tables 2-3, 2-4 and 2-5.

Use of parapets walls can be considered in embankment dams, on a case to case basis.

RESERVOIR WATER LEVEL	CONCRETE DAM	EMBANKMENT DAM
Normal Water Level	$H_f = h_w + h_c + h_a$ $H_f \geq 2.00 \text{ m}$	$H_f = h_w + h_c + h_a + 1$ $H_f \geq 3.00 \text{ m}$
Surcharge Water Level	$H_f = h_w + (h_c/2) + h_a$ $H_f \geq 2.00 \text{ m}$	$H_f = h_w + h_c + (h_a/2) + 1$ $H_f \geq 3.00 \text{ m}$
Design Flood Water Level	$H_f = h_w + h_a$ $H_f \geq 1.00 \text{ m}$	$H_f = h_w + h_a$ $H_f \geq 2.00 \text{ m}$
$H_f$ : Freeboard, $h_w$ : Wave height due to wind, $h_c$ : Wave height due to earthquake $h_a$ : Allowance for gate operation		

Table 2-3: Minimum freeboard for Dams of Japan (ICOLD, 2017).

DAM TYPE	CONCRETE DAM	EMBANKMENT DAM	
Dam Height	Any Height	15.00m	$H \geq 90 \text{ m}$
Freeboard	1.00 m	1.50 m	3.50 m

Table 2-4: Minimum Freeboard for Dams in Italy (ICOLD, 2017).

DAM HEIGHT	CONCRETE DAM	EMBANKMENT DAM
$H \geq 40 \text{ m}$	1.00 m	2.00 m - 3.00 m
$10 \text{ m} \leq H < 40 \text{ m}$	1.00 m	1.50 m - 2.00 m
$H < 10 \text{ m}$	0.50 m	1.00 m

Table 2-5: Minimum Free board for Dams in Switzerland

## Parapet walls

They are in general vertical and are connected with the impervious zone of a given dam. For modifications in existing dams, parapet walls should only be used to provide freeboard for wave run up. Also the top level of the core i.e. hearting of an Embankment dam should be higher than the revised MWL plus wind setup. When parapet walls are considered to be a part of freeboard allowance, the following measures should be taken (US Bureau of Reclamation, 2012).

- The ends of the parapet wall must be adequately tied into the impervious zone and the abutments of the embankment dam to avoid excessive seepage or scour beneath the wall. It is not always necessary to embed a wall deep into the dam's core if the wall can be tied into other embankment materials adjacent to the core that are impervious enough to provide a good seepage barrier, are resistant to erosion, or if the exposure to floodwaters is very short.
  - Provide proper zonation around and beneath the parapet, including an adequate tie into the impervious zone, if necessary, to prevent undercutting and erosion
  - Future foundation and embankment settlement that would adversely affect the structural integrity of the parapet wall must be accounted for in construction sequencing or the design of the parapet wall.
  - The parapet wall must be designed to withstand hydrostatic and hydrodynamic (wave) loads.
  - Drainage off the crest around or through the wall must be provided.
  - Joining and sealing the wall units together with each other and each end of the dam shall be accomplished
  - Safety and security must be ensured.
- Maintenance, snow and ice removal, sight lines, and aesthetics issues should be addressed.

If evaluating existing parapet walls, note how the walls are founded and tie them with the abutments and the impervious zone of the dam. Some existing parapet walls only extend to the ends of the dam, leaving an opening for floodwaters or wave action to concentrate around the ends of the wall where the camber may be the least, and erode the embankment dam along the groins. This should be corrected.

Other factors that influence freeboard:

- a) Floods: IDF Hydrographs, peak flow, duration of the flood, reservoir storage capacity, spillway and outlet work discharge capacities and reservoir operation. (US Bureau of Reclamation, 2012).
- b) Reservoir Operation: Seasonal fluctuation of the reservoir level is of fundamental importance along with wind intensity. Remoteness of the dam site and conditions downstream for safety are also important parameters.
- c) Mal functioning of the Spillway and Outlet Works: Operation and maintenance of the appurtenant works i.e. Gates and Hoists of the Spillway and Outlet works is an important factor for freeboard requirements. Malfunctioning of the gates, either due to operation error, mechanical or electrical failure, blocking of the works by floating debris, could cause the reservoir to rise over maximum water levels.
- d) Ungated spillways: This type of structure may be vulnerable to clogging by floating debris in cases where spillways piers reduce the span length of the ungated weir. Also, for small diameter morning glory type of spillways the floating debris can reduce its discharge capacity. For these two cases, the reservoir level would rise and freeboard will be reduced.
- e) Gated Spillway: Even if the facility is maintained well, and there is adequate at-

tendance by an operator, the malfunctioning operation of a spillway and or outlet gate by human error or by mechanical/electrical failure, should be recognized. This is particularly critical for dams with small reservoirs, where failure of one or two gates, may cause a rapid rise of the reservoir level and a subsequent loss of freeboard. In that case, sensitivity analysis should be performed with one, two, three gates inoperative to evaluate the resulting freeboard.

### **Freeboard of Concrete Dams**

Freeboard of an existing concrete dam is not as critical as it is for an embankment dam because the overflow is not likely to wash away the concrete dam. In case of overtopping of a concrete dam, failure will depend on the ability of the abutments and foundation to survive the force of the water flowing over the concrete dam. The geology of the dam site should be carefully examined by engineers and geologists to make a judgement on the potential of erosion of the material. If erosion of the abutments and foundations lead to undercutting of the concrete structures, failure may result.

### **Parapet walls for concrete Dams (USBR Practice)**

A standard 3.5 feet (1.1m) high parapet wall provides all of the freeboard that is required for concrete dams. This wall is intended to keep waves from moving over the dam during high reservoir water levels.

The freeboard requirements for Concrete/Masonry dams in India are discussed in IS 6512.

### **Freeboard at existing Indian Dams**

The freeboard in existing Indian dams is generally checked based on IS 10635 and IS 6512. The procedure given in the Manual for Assessing Structural Safety of Dams (CWC, 2020) can also be referred to. The minimum freeboard allowed in Embankment dams is 1.5 m and in case of Concrete/Masonry dams it is 1.0 m. At times the solid upstream

parapet wall is also considered to be a part of freeboard allowance in existing dams. It is also checked that the impervious core of the Embankment dam is above the revised MWL for the revised flood.

### **2.3.4 Vulnerability-Overtopping**

Inadequate spillway capacity is a common problem with many dams. Many dams worldwide also do not have adequate spillway capacity.

Chapter 3 presents data about dams in India and spillway capacities.

Many early dams were designed for floods based on empirical formulae or local historical records or even a limited number of measurements. Over the years with the technological advances and availability of more data, there have been improvements in the analysis of extreme flows and, tools for evaluating the hydrological events.

Designers and dam safety engineers should fully evaluate all options available when the dam overtopping is a possibility. While choosing an alternative, the one that avoids flow over the dam crest has been the traditional and safest approach. However, providing project-specific protection during dam overtopping can be a viable method in some instances to safely convey large flows downstream of the dam. A major concern with overtopping protection is that if protection fails during a flood event and the underlying material of the embankment becomes exposed, erosion and head cutting in the embankment may progress very rapidly and eventually lead to failure of the dam.

A decision to use dam overtopping protection instead of improving the service spillway or constructing an auxiliary spillway or raising the dam crest elevation, or imposing a reservoir restriction should be made with careful considerations of all potential impacts.

Overtopping may also be induced by malfunctioning of the spillway or bottom outlets gates, or by spillway with difficulties in passing design discharge due to obstructions by floating debris, or in the upper latitudes by ice formation or even combination of these two large problems, in many areas of the planet. Reservoir inflows during storms with volumes typically larger than normal, or, in the case of badly operated reservoirs can result in the dam being overtopped.

**SEE APPENDIX A**

To identify the possible failure mechanisms caused by the overtopping of concrete or embankment dam refer to Failure Modes FM-1, FM-2 and FM-3

### Assessment of the Vulnerability of Overflowing embankments

Laboratory tests of overtopping flows for various embankment slopes have demonstrated that scour starts near the top of the dam, with supercritical depth at the dam crest developing at a very close location, and continues till the toe of the dam.

Some experiences based on model and prototypes of overflowing embankment have permitted to reach the following conclusions:

- Uniform vegetation can generally provide some protection for shallow overtopping depths (up to about 0.3 m) for short duration and few hours specially in clayey compacted soil surfaces
- Granular rockfill materials at the embankment toe may be more easily eroded and can cause undermining of a more resistant cohesive fill
- High tail water reduces the head differential on the embankment and can reduce erosion
- Interruptions to a smooth downstream slope surface (e.g., a change in slope [either from steeper to flatter or from flatter to steeper] of a projected structure, berm, roadway or abutment groin) pro-

duce turbulence which can initiate erosion and accelerate breaching.

- Flow concentration due to elevation changes along the embankment crest (generally caused by camber of the crest or by crest settlement) can initiate erosion.
- Flatter embankment slopes have a greater resistance to erosion.

### 1. Embankment Dam Overtopping Protection Considerations

Overtopping protection should not be considered a low-cost alternative to substitute the service spillway, especially where it is of frequent use, high unitary discharge and high head, where large volumes are retained by the dam and where large population is located in the downstream reaches of the reservoir. Figure 2-22 shows an example of dam failure due to inadequate spillway coupled with inoperable spillway gate due to electric power shortage. Also see Figure 2-23.



Figure 2-22: Overflow on an Embankment dam.

Overflow embankments can work as an auxiliary spillway with service spillway provided to pass the frequent floods. Planning the use of an overflow embankment as an auxiliary spillway should consider at least the following issues:

- Flows with significant discharge intensity may be required to pass over the erodible material.

- Higher static loading on an embankment dam may result in a slope failure.
- Uncontrolled leakage from the overtopping protection could cause embankment erosion and instability.
- Debris carried by the flow can damage the overtopping protection.
- Numerous overtopping projects have been constructed; few have operated satisfactorily at low discharges, none have worked with design flows.
- Overtopping materials usually create a visual change of appearance of the structure.
- Depending on a number of other considerations with regard to how the flow is impacting the toe of the dam, the energy dissipation arrangement should be planned/ designed.



Figure 2-23: Tous Dam (Spain) failure due to overtopping in the flood of 1982.

### Site investigations and analysis of Overtopping Protection

Should include at least the following studies:

- Site reconnaissance.
- Sub-surface investigations.
- Slope Stability analysis.
- Foundation Analysis.
- Seepage analysis.

### Selecting the Type of Overtopping Protection Systems for Embankment Dams

Should include consideration of the following factors:

- Unit discharge
- Maximum head on crest
- Embankment or drop height
- Embankment materials
- Downstream slope
- Flow duration
- Flow velocity
- Shear stress
- Surface discontinuities that can lead to irregular hydraulic flow patterns or turbulence
- Potential for differential settlement
- Cavitation potential
- Erosion potential (or resistance to abrasion) and freeze-thaw damage
- Energy dissipation - Downstream channel conditions
- Downstream consequences
- Constructability
- Maintenance requirements
- Potential vulnerabilities including terrorism and vandalism,

Design considerations for overtopping protections require more rigorous and detailed analysis to ensure stability for larger unitary discharges and large drops and velocities.

**SEE CHAPTER 3**

**Includes the topic about spillways over embankment dams.**

### Remediation/Rehabilitation measures

The design and construction of overtopping protection measures for dams is increasingly

being viewed as a viable alternative to constructing larger spillways or increasing the dam height by raising the dam crest. The decision to pursue overtopping protection for a dam must give strong consideration to the risk of failure of the protection system, which could lead to a full breach of the dam. Overtopping protection should generally be reserved for situations with a very low annual probability of operation, and with physical or environmental constraints and a prohibitive cost of other flood protection alternatives.

Alternatives for overtopping protection may utilize a variety of different materials, such as roller-compacted concrete, conventional cast-in-place concrete, precast concrete blocks, gabions, reinforcement mats, vegetative cover, flow-through rockfill, reinforced rockfill, riprap, and various types of geosynthetic materials. Not all materials are applicable in every situation. Significant research and hydraulic testing has been conducted on these materials but since most overtopping protections are designed to function at an infrequent recurrence interval, practical experience on constructed projects that have been subjected to overtopping flows is limited.

New design approaches have been developed that allow for the dam to be safely overtopped, that should be compared with costly measures such as raising the dam height or building an additional spillway.

It is emphasized however, that the decision to pursue overtopping protection must also consider the risk of failure of the protection system which may lead to breaching of a dam. This is particularly true for embankment dams in the sense that any significant mistake could lead to catastrophic failure once the dam is overtopped.

### **Measures for Rehabilitation**

Erosion and slope instability resulting from overtopping flow is the principal cause of failure for embankment dams. The poten-

tial impact of the proposed modifications must be evaluated. Any reduction to the embankment cross-section can decrease the factor of safety for slope stability, especially due to excavation required during construction. Excavation at the toe of the embankment to construct the various features of the overtopping protection, in particular for construction of a downstream stilling basin or -steepening of the downstream slope, will change the stability of the embankment and could increase the potential for internal erosion. An evaluation of the estimated risks of dam failure during construction should be performed as part of the design of overtopping protection for an embankment dam.

### **Roller-Compacted Concrete**

The development of RCC technology has provided a successful method of erosion protection for embankment dams, which has proven to be cost effective while affording a number of other advantages, including very rapid construction with minimal project disruption. In most cases, construction for overtopping protection is limited to the dam crest and downstream slope, with no requirement for any reservoir restrictions. RCC spillways generally consist of non-air-entrained concrete, without reinforcement, water-stopped joints, or anchorage, but with underdrain systems similar to conventional concrete spillways. For hydraulic performance of stepped spillway of RCC see Chapter 3.

Some beneficial aspects of RCC used as material for overflow protection of an embankment dam are:

- RCC is a material suitable for a wide range of flow depths and velocities
- RCC has exceptional resistance to cavitation, erosion, and abrasion damage from both high and low velocity flow
- RCC can generally resist debris impacts (such as trees, cobbles, and boulders) without significant damage.

- An average spillway discharge for RCC embankment spillway tested in prototype can be as large as 2.4 m<sup>3</sup>/s/m, for a flow overtopping depth up to 1.5 m.
- RCC is typically placed in horizontal lifts on the downstream slope, resulting in a stepped chute.
- Stepped chutes significantly increase the rate of energy dissipation on the downstream face of the dam compared to a smooth spillway
- RCC steps reduces about 73 percent of the energy available, depending upon step height, flow depth, and other factors, residual energy and thus reducing the size of an energy dissipater and the probability of scour.
- A minimum 2.5 m width is normally required for the horizontal lift surface to operate standard placing and compacting equipment. Depending on the slope of the embankment, this provides an effective concrete vertical step of about 0.7 to 1 m.
- Seepage through RCC lifts can be designed to be safely handled by a properly designed drainage system beneath the sloped RCC chute
- RCC spillway crests that follow the shape of the embankment crest to simplify construction represent a broad-crested weir having a low coefficient of discharge. By increasing the efficiency of the spillway crest section the required crest length of the spillway and/or the flow depth can be reduced.
- An upstream cutoff wall is generally recommended to increase the seepage path between the RCC approach apron and soil interface.
- The downstream apron or stilling basin is a critical feature of an RCC spillway located on a dam embank-

ment, and must be designed based on the flow depth and incoming velocity, unit discharge, operating frequency, tail water conditions, foundation conditions, erosion control requirements, and downstream consequences.

- A downstream cutoff wall should be sized to prevent undermining of the spillway from channel erosion.

### Conventional concrete

Overtopping protection for embankment dams utilizing conventional reinforced concrete generally relies on a continuous layer of concrete to serve as the flow surface for reservoir releases and to protect the underlying embankment from high velocity flows.

Some features of this option for overtopping protection on embankment dams are:

- Guide wall normally provided at the sides of the overtopping protection to contain the overtopping flows and protect the abutments.
- The concrete slabs generally have a minimum thickness of 0.3 m and include reinforcing steel.
- Excessive settlement of the underlying embankment may affect the structural integrity of the concrete by causing cracking or offsets at joints (as may also be the case for RCC).
- Stagnation or uplift pressures can cause catastrophic failure of the concrete overtopping protection as a result of water flowing into open joints and cracks during reservoir releases.
- If water entering a joint or crack reaches the concrete bedding materials or embankment surface, failure can result from excessive uplift pressure on the concrete slab and/or by erosion of the underlying materials.
- If the drainage system is inadequate and the slab is insufficiently restrained, the uplift pressure can cause hydraulic jacking and progressive

loss of the concrete overtopping protection.

- If drainage arrangements are provided without adequately designed filters, erosion of foundation material is possible and structural collapse may occur.
- Concrete deterioration resulting from delamination, poor consolidation, alkali-silica reaction (ASR), freeze-thaw damage, frost heave, and sulfate attack can exacerbate this potential failure mode by initiating cracks and concrete spalls, opening joints, creating offsets into the flow, and causing separation of the slab from the foundation.
- Defensive design measures include joint water stops and keys, transverse cutoffs, reinforcement crossing transverse joints, soil anchors, filtered underdrains, and rigid plastic foam insulation. See figure 2-24.

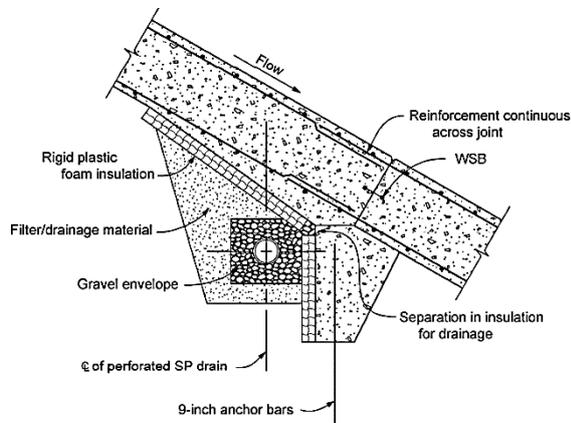


Figure 2-24: Defensive design measures for concrete overtopping protection slab (Fielder, 2016)

- Cavitation damage can occur if the water pressure is reduced locally because of an irregularity in the flow surface. An air slot or ramp can be provided to introduce air into spillway flows at critical locations to reduce the potential for cavitation damage to concrete.

### Precast Concrete Blocks

Pre-cast blocks are used over earthen materials to provide a hard surface for overtopping flow to pass safely without eroding the underlying earthen materials. It is comprised of a matrix of individual concrete blocks placed together to form an erosion-resistant revetment with specific hydraulic performance characteristics (see Figure 2-25). Of primary importance for overtopping protection is to select a commercial product that has been tested under the flow conditions expected during overtopping, and to ensure that an adequate filtered drainage layer is provided beneath the block system. Typical applications may experience high flow velocities, moderate flow depths, hydraulically steep slopes, and energy dissipation on the flow surface.



Figure 2-25: Placement of cable tied mats over a geotextile on the downstream face of Strahl Dam, Indiana, USA

### Vegetation and Turf Reinforcement

During overtopping flow, vegetation can also provide protection against the initiation of concentrated erosion that leads to head cut development and dam breach (see Figure 2-26). For larger flow rates, vegetation may delay breaching sufficiently to permit evacuation of downstream areas. Some aspects associated with vegetative cover as a protection are:

- Good maintenance of the cover is essential to achieve significant protective benefits.
- Vegetation is not suitable for very steep embankments because of the difficulty of performing mowing and

other maintenance required to achieve uniform cover.

- Installation costs for vegetation are often lower than for other forms of overtopping protection, but maintenance costs can be higher and performance may be limited.
- Vegetation provides protection to an embankment in two functional ways: (1) protection of the soil surface by reduction of velocities and stresses at the embankment boundary as a result of the coverage provided by stems and leaves that lay down in the flow and blanket the surface; and (2) the reinforcement of the underlying soil due to the presence of plant roots.
- The reinforcement aspect may be further improved by the use of turf reinforcement mats that can improve root mass continuity following full vegetation establishment.



Figure 2-26: : Vegetated embankment experienced overtopping flow.

### Riprap

Riprap on the downstream slope has been recognized to have some capacity to prevent the initiation of embankment erosion during overtopping flow (see Figure 2-27). Riprap is generally composed of high quality crushed or quarried rock (typically granite or limestone) with relatively uniform size. Some aspects associated with Rip Rap as overflow protecting material are:

- Flow is conveyed through, and in some cases above, the riprap layer installed over bedding.

- Rip rap prevents erosion by reducing flow velocities and hydraulic stresses directly against the surface of the erodible embankment materials.
- Riprap is relatively economical compared to other options and is a popular slope protection option for arid areas and on steeper embankment slopes where vegetation is difficult to establish and maintain.
- As overtopping protection, riprap is most cost effective for lower flow rates and flatter slopes, which do not demand extremely large rock sizes
- Riprap has been specified for the protection of small, low hazard dams, but at this time there are no known applications of riprap specifically designed for overtopping flows on significant or high hazard dams.
- Good quality control of materials and installation procedures are essential for obtaining good riprap performance. Specifications can be difficult to maintain during production, especially as rock sizes increase.
- It is especially important to maintain cleanliness of materials and prevent size segregation during handling and placement.
- Since a large fraction of the flow is conveyed within the riprap layer, long-term performance could potentially be affected by infiltration of fine materials into the riprap layer (e.g., sediment or vegetation).
- Degradation of rock over time due to weathering can also affect long term performance, but this should not be an issue if high quality materials are used.
- Flow transition areas at the toe, crest, and groins are potentially vulnerable, although testing that has included crest and toe areas has shown thus

far that failure of the riprap will occur first on the slope.



Figure 2-27: Rock chute spillway of Little Washla Site, USA.

## 2. Protection of concrete dams subject to Overflow

### General

The basic types of concrete dams are gravity, arch, and buttress dams. Figure 2-28 and 2-29 show two examples of gravity dams during overflow in India. Potential concerns for overtopping of concrete dams of all types generally involve blocky or erodible rock abutments or foundations, rather than concerns for the dam structure itself. In these cases, overtopping protection may be required for the exposed abutments and foundation within the impact zone of the overtopping flow, to prevent the loss of materials and subsequent undermining of the dam which could otherwise result in instability and failure.

Also, higher hydrostatic loads on concrete dams resulting from the passage of a flood event due to increase in Maximum Water level (MWL) could produce lower factors of safety for sliding at a lift line within the body of the dam, at the dam foundation contact, or along a potential slide plane within the foundation, requiring some form of concrete buttress or reinforcement. Generally overtopping protection for concrete dams must be very robust since the system should be capable of withstanding the impact of concentrated jets/flows overtopping the dam that may have a significant fall height. The following are the most commonly used over-

topping protection systems for concrete dams.



Figure 2-28: Sardar Sarovar Dam (India) during the monsoon flood.

### Roller Compacted RCC

In addition to overtopping protection to the dam foundations, RCC can also be used for construction of a massive downstream buttress for the dam to improve sliding stability, as per site-specific requirements.

RCC buttresses have been constructed by US Bureau Reclamation for a straight masonry gravity dam (Camp Dyer Diversion Dam in Arizona), for a curved concrete arch dam (Santa Cruz Dam in New Mexico), and for a concrete overflow spillway structure (Pueblo Dam in Colorado).

RCC may also be used to protect the dam foundation from erosion and head cutting from an impinging jet/flow, but would not lend itself to the protection of steep abutments.

Some features of RCC acting as overflow protection material for concrete dams are:

- RCC is to be placed in horizontal lifts along the downstream face of the existing dam structure (if required) to improve the stability of the structure by resisting the additional hydrostatic loads in flood discharging condition on account of higher MWL.



Figure 2-29: Poringalkuthu Dam (India) before, during and after the overflow.

- The downstream face of the RCC buttress (if provided) can be stepped to provide energy dissipation of the overtopping flow, reducing the design requirements for the terminal energy dissipation structure.
- Drainage pipes may be required at the foundation and structure contact surfaces to collect future seepage and relieve potential uplift pressures. Contraction joints can be provided for crack control.
- General construction considerations for RCC buttresses are similar to those for other types of RCC construction.
- An RCC buttress for a concrete dam will not require upstream forming, but will require special surface preparation

and treatment for the upstream contact surface, which may consist of cleaning using a high-pressure water jet, and the use of a special concrete mix to ensure bond between the RCC and the existing structure, without mechanical anchorage.

- If the RCC buttress is constructed against a sloping concrete dam face, the buttress width may be fairly constant for the full height of the structure.
- Sufficient sliding resistance due to friction and cohesion must be provided by the buttress at the lift lines. For Pueblo Dam, high strength rock bolts were used to reduce the tensile stresses that could develop in the RCC buttress, and to provide additional active resistance across the foundation failure surface.
- RCC will also be required to be used to protect the dam foundation from erosion and head cutting from an impinging jet as for conventional concrete, but would not lend itself to the protection of steep abutments.

### Vulnerabilities and Risk of RCC as overflow protection

Potential failure modes for RCC overtopping protection for a concrete or masonry dam could include:

- Undermining of the downstream end of the RCC protection due to inadequate energy dissipation resulting in erosion or scour within the outlet channel
- Inadequate coverage of RCC protection, resulting in erosion or scour of the foundation due to impact from the overtopping flow
- Deterioration or cracking of the RCC protection, resulting from poor compaction, freeze-thaw damage, or thermal stresses
- Inadequate bond at lift surfaces, resulting in insufficient sliding re-

sistance. Proper design and construction methods should ensure that these or other potential failure modes do not represent an unacceptable risk to the completed structure.

### Conventional concrete as overflow Protection

Conventional or mass concrete can be used to provide overtopping protection in the form of concrete overlays that protect the underlying rock foundation at the downstream toe of the dam and along the downstream abutment (see Figure 2-30).

The overlays protect the rock from overtopping flows that could pluck rock blocks from the rock foundation or that could scour and remove material along shears or faults within the dam foundation. Concrete overlays built to protect the dam foundations from overtopping flows that flow down the dam to the river channel may be seen in Figure 2-31.



Figure 2-30: Concrete overtopping protection for Santa Cruz Dam.

Conventional concrete design as used for overtopping should include the assessment of the load:

- a) **Impinging jet Impact** load: Impact loads from impinging jets may induce compressive, shear, and bending stresses in protective slabs. Flow aeration and reducing the angle of impingement will reduce the actual pressure on the foundation.
- b) **Uplift due to Impingement Jet:** Impinging jets entering open joints

in the foundation or open cracks in a protective slab may develop local uplift pressures equal to the full water head at the location if the foundation is not adequately drained.

- c) **Steady state Uplift:** Seepage under reservoir head will produce an uplift pressure distribution between the upstream face of the dam and the downstream end of the protective slab. The protective slab should be designed to resist the maximum loads from the uplift pressure distribution, but generally not less than 10 feet of head.

### Hydraulic studies for Safety Assessment

Hydraulic studies including physical models of the overtopping flows will be needed to ensure that the coverage of the overlays is adequate. The studies will provide details of the jet loads/pressures, jet trajectories, places where the jet will impact (inner jet) the different areas of the foundation, and the areas to be protected including the adequacy of the energy dissipation arrangements planned.

### Vulnerabilities and Risk of Conventional Concrete as overflow protection

A series of typical vulnerabilities include, updated frequency floods, deterioration of concrete protections, efficacy of drainage systems at the interfaces, inaccurate prediction of jet trajectories etc. Figures 2-32 to 2-35 below show Hydraulic Investigations and Physical Model studies of the Erosion Potential of Flows Overtopping Gibson Dam (taken from Hydraulic Laboratory Report HL-2006-02, USBR, April 2006).

Overflow protection materials along with detailed hydraulics information is contained in the “Technical Manual-Overtopping protection for Dams Best practices for design, construction, problem Identification and Evaluation, Inspection, maintenance, Renovation, and Repair”, (2014), FEMA, 2014.



Figure 2-31: Concrete overtopping protection at the downstream toe of a dam



Figure 2-32: Photo showing Gibson dam and right abutment protection concrete

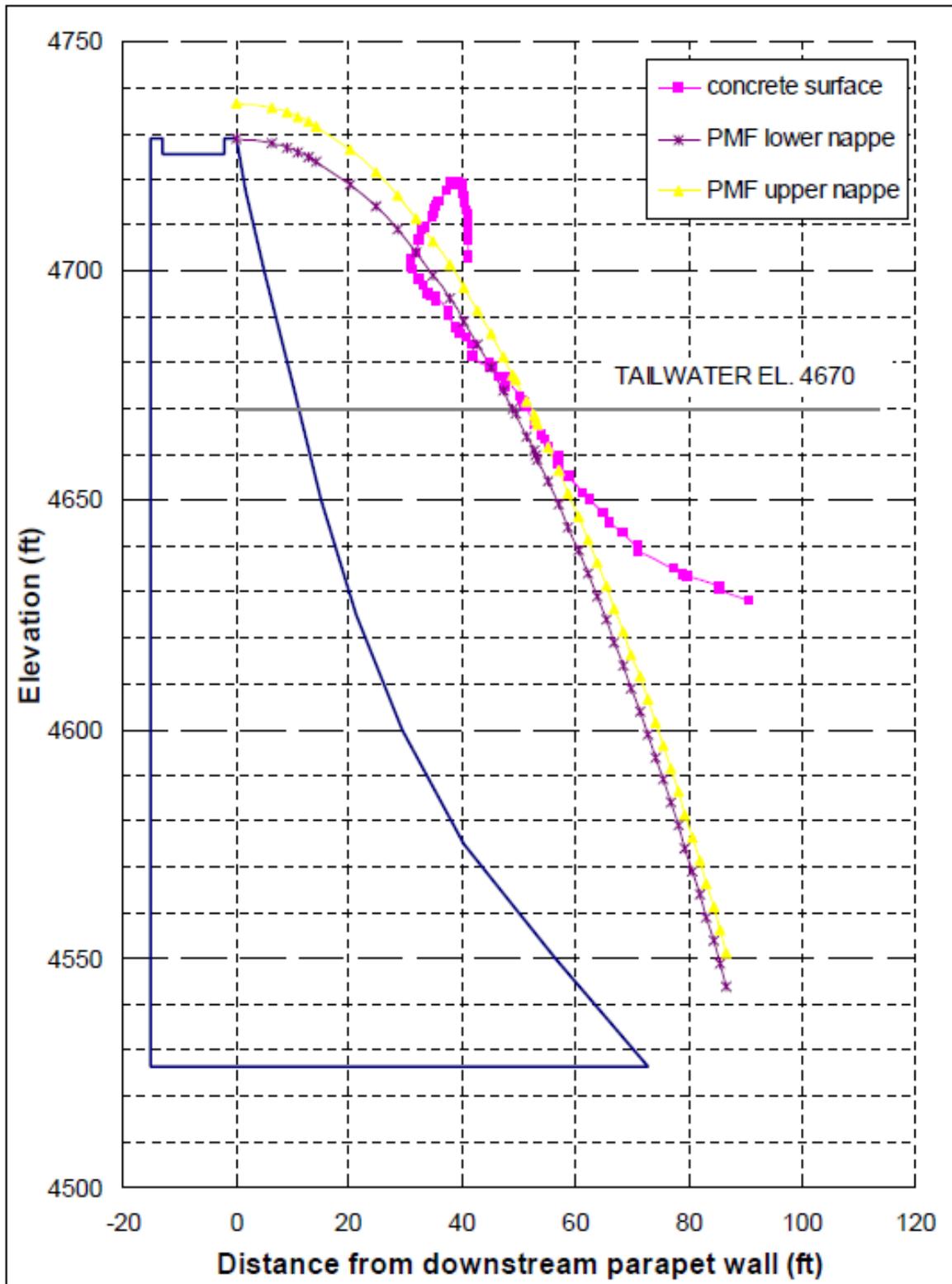


Figure 2-33: Sectional view of the final trajectory profile for the PMF for Gibson Dam through dam section aligned with the river channel.

Note: that the concrete surface is referring to right abutment protection concrete



Figure 2-34: Footprint of the trajectory with no spread of the jet for the PMF overtopping at Gibson Dam

Note: the location of the footprint extends beyond the right abutment protection between contour elevations 4660 and 4710. The tailwater for the PMF is shown on the plan view in blue at El. 4670

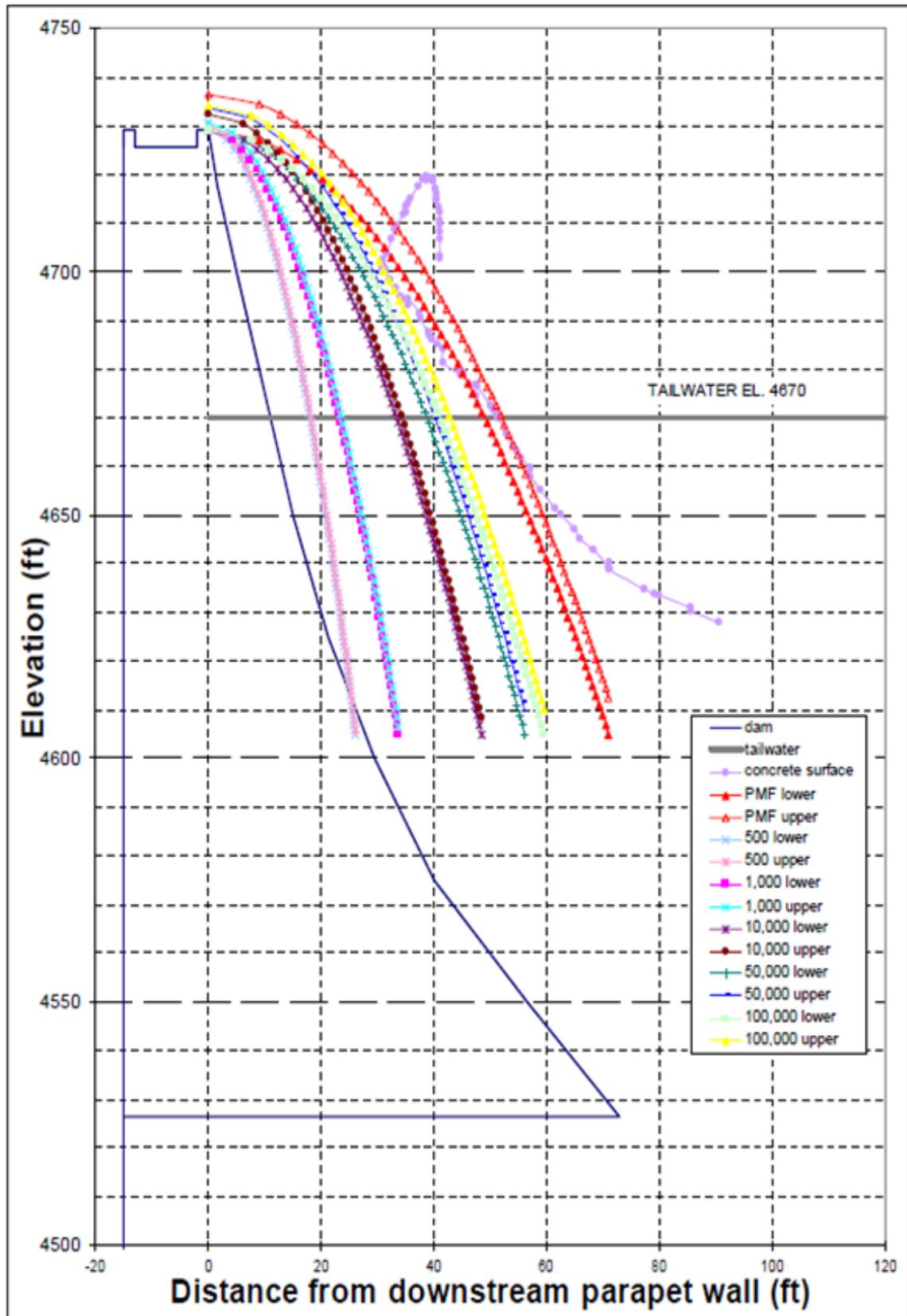


Figure 2-35: Sectional view of predicted trajectories for various flood frequency (PMF and various frequency return periods) overtopping flood events at Gibson Dam

### 2.3.5 Various Structural and Non-Structural Mitigation measures considered to take care of the increase in design flood in a dam

#### Structural Mitigation Measures

- Raising the height of the dam to provide for freeboard necessary above the higher MWL
- Raising the height of gates by lowering the spillway crest.
- Constructing one or more additional (auxiliary) spillways, fuse plugs with breaching section, flush bars, etc.
- Provision of a solid parapet wall on the upstream at the dam top (where not available) provided that it is able to provide for the revised freeboard requirement.
- Strengthening the crest and downstream face of the embankment to allow for some overtopping.

#### Non-structural Mitigation Measures

- Improve methods of collecting data to give advance warnings of adverse conditions and to monitor the response of the dam and reservoir
- Improve operation of the reservoirs by lowering the reservoir level for conservation purposes to increase flood control volume
- Modifying river basin flood characteristics by building flood detention devices or building an upstream dam
- For reservoirs in cascade, use upstream reservoir capacity to reduce flood peaks downstream.
- For cascade reservoirs, integrate operations of reservoirs to diminish peak flow downstream and avoid possibility of flooding

### 2.3.6 The Approach Channel

When spillways are placed independent of the dam works they may require an approach channel whose function is to connect the reservoir with the spillway control structure, unless the control structure is located inside of the reservoir or in the border of the reservoir (Bolinaga, 1999). The approach channel conveys water from the reservoir to the inlet structure or to the control structure (USBR, 2014). Excavation will be required to construct the approach channel. Individual spillways, located outside of the reservoir and/or away from the dam works may need streamlined approach, normally enhanced by additional transitional structure built to improve flow hydrodynamics. Figure 2-36 to 2-39 may be seen for reference.



Figure 2-36: Approach Channel of the 800 MW Sogamoso Hydropower Station, Sogamoso River, Colombia. Contemplates a abutment excavation of 200 m height (Cortesy ISAGEN)



Figure 2-37: Approach Channel for the ungated El Guapo Spillway Dam, Venezuela, (Photo A Marcano)



Figure 2-38: Approach channel and transition structures to connect with the ungated control structure, El Guapo Dam Spillway, Venezuela,. Scale Model 1:2, Edelca Hydraulics, Lab. (Photo A Marcano)

The approach channel delivers flow to the control structure, and its geometry, in both elevation and plan calls for an smooth transitions with the purpose of guarantee tranquil subcritical flow, with smooth behavior and minimum energy losses.

Good practice in the design of the Spillway approach channels include:

- Maximum flow velocities 0,5-1 m/s, to avoid energy losses and flow erosion.
- Normally approach channel contours are not lined.
- Subcritical tranquil flow, little turbulence.
- Provider of gradual transitions in both the abutments and the approach channel bed elevation.
- Bed elevation design in terms of excavation cost and hydraulic performance.
- Bed slopes horizontal or slightly adverse.
- Trapezoidal, most common cross section.
- Stable banks, either guaranteed by a competent geology, or by bank slope stabilization works.

Vulnerability aspects	Consequences	Measures recommended
High approach velocity	High energy loss, perturbed flow, presence of Vortex, circulations, waves, low discharge capacity coefficient,	Widen/deepen approach cross section in both plan and elevation, use transitional structures, streamlined abutments and/or piers
Loss of flow cross section by bank sliding	Increase of reservoir elevation during floods, low discharge capacity, non-uniform flow approaching the control structure, abrasion on concrete and gate structures by materials drag by the flow,	Use stabilization works for unstable banks. Avoid rapid reservoir draw-down. Increase number of bays.
Loss of flow cross section by floating debris/ice	Blockage of the flow cross section, spillway gate malfunctioning, increase of reservoir elevation, overtopping, even dam failure	Increase dam free board (particularly in the case of embankment dams), placing of trash racks at some distance of the control section, provision of barriers to collect debris in the reservoir area, provision of heated devices to reduce/avoid gate jamming , increase number of bays, redirect debris from spillway flow path. See Figure 2-39.

Table 2-6: Vulnerability aspects, consequences and measures recommended

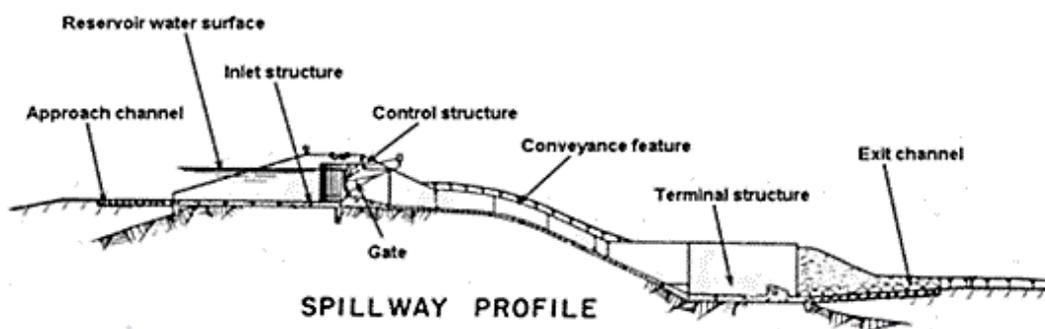


Figure 2-39: Spillway Profile

Aspects to define	Tool for assessment
Geometry , Plan and Elevation	Numerical /Physical Model 2D/3D
Alignment of approach channel	Numerical/Physical 3D
Abutments	Numerical
Flow conditions in the vicinity of the control section	Numerical/Physical 3D
Vorticity vs gate opening	Numerical/Physical 3D
Behavior of floating debris and Ice	3D Physical Model
Influence of the sliding material on discharge capacity	3D Physical Model
Transitions between approach channel and control structure	3D Physical Model

Table 2-7: Approach Channel Assessment

### Approach Channel Vulnerability

Approach channels can reduce its discharge capacity by different reasons:

- Poor design: high velocity, poor discharge coefficient, approach flow severely accelerating, presence of waves, free subsurface perturbations, vortex, circulations
- Loose of flow cross section by lack of bank stability.
- Loose of flow cross section, gate malfunctioning by floating Ice/debris.
- Gate malfunctioning due to ice jams, low temperature.

**SEE APPENDIX D**

**Hydraulic Modeling**

## 2.4 Measures for Increasing the Reservoir Storage

### 2.4.1 Reservoir

It is observed that in smaller reservoirs used for power generation, daily fluctuation in reservoir levels are unavoidable due to peaking requirement and limited poundage. The reservoir bank line falling in unstable zones may need to be protected by providing a retaining wall or gabions protection with free draining backfill to avoid further landslides/subsidence due to peaking operations.

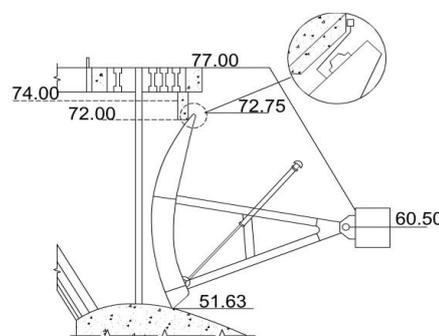


Figure 2-40: Mask beam used to raise by 2m the reservoir of Tucuri Dam, Brazil (Erbiste, P, 2004)

### Increasing Storage capacity by Heightening of gates

Need for increasing the storage volume of reservoir for different reasons may be accomplished by raising of the height of spillway radial gates, which represents an advantage for projects where radial gates are used (Erbiste P., 2009) . The associated hydrostatic pressure on the submerged gates, however, increases and it is necessary to recheck the design of the gates and their hoists, because the hydrostatic force increases accordingly. In that case, there are several options: Substitute the existing gates by new higher gates, Heighten the existing gates, install new gates above the existing ones or even place new gates on top of the dam i.e. over spillway crest.

#### Heightening of the existing gates

Heightening of the existing gates can be achieved by either:

##### Top Heightening (Figure 2-44)

- Is accomplished by the installation of a new gate panel on the top of the gate leaf
- It can be an extension of the skin plate and vertical beams or a flap gate hinged in bearings mounted on top of the existing structure
- The gate structure has to be designed and reinforced for the additional hydrostatic load and the design of civil structures are also to be checked.

##### Bottom Heightening (Figure 2-41)

- Involves the addition of a new gate panel in the bottom of the gate leaf
- Loads of the original gate structure practically remains unchanged.

##### Mask Beam (Figure 2-40)

- Consists of the construction of a structural beam just upstream and next to the top of the gate leaf
- Includes a top sealing between the skin plate and the upper beam.

##### Crest Heightening (Figure 2-43) Not a favored option.

- It envisages addition of concrete on the existing spillway crest.
- Dismantling of the spillway at crest will be involved resulting in instability of existing spillway piers, bridge etc.
- May result in instability to the spillway structure due to vibrations involved in dismantling. This may also aggravate seepage problems in the dam. No need to reinforce/modify the gate structure
- Amount of civil works are considerable. Civil designs may be complicated depending on crest shapes, spillway profile etc.
- Reservoir level should be low to execute the works

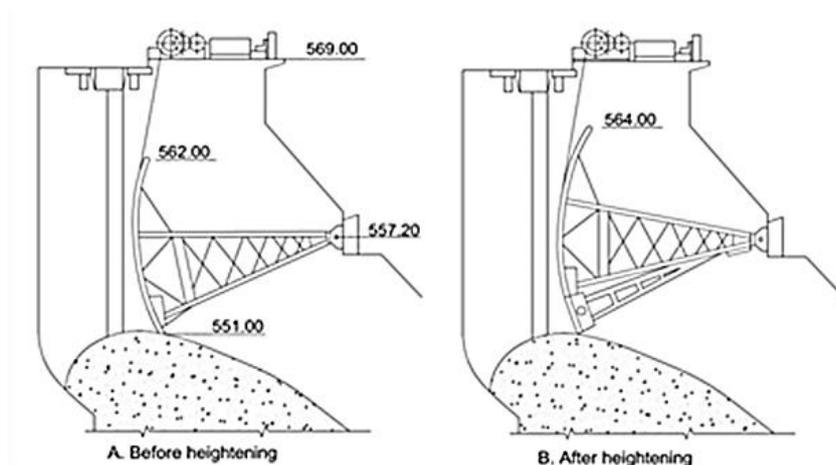


Figure 2-41: Bottom heightening of the Castro Dam radial gates, Spain. (Erbiste, P., 2004)

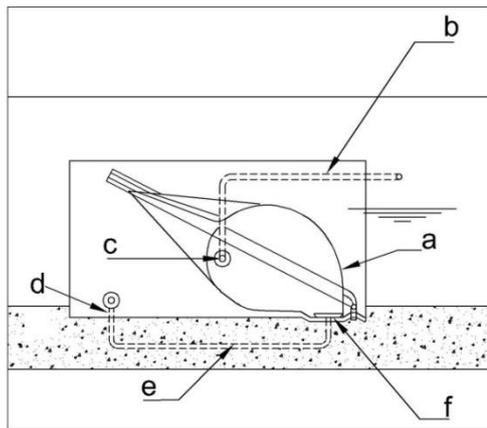


Figure 2-42: Inflatable gate (Sumigate)

- a) Rubber Body;
- b) Air Supply-exhaust;
- c) Air Supply-exhaust opening in body;
- d) drain pipe outlet;
- e) drain pipe;
- f) drain pipe intake

### Installation of new gates on the top of the dam

This case includes dams with uncontrolled spillways. In such cases, the increase of the water level can be accomplished by the installation of either metal gates or inflatable gates over the spillway crest.

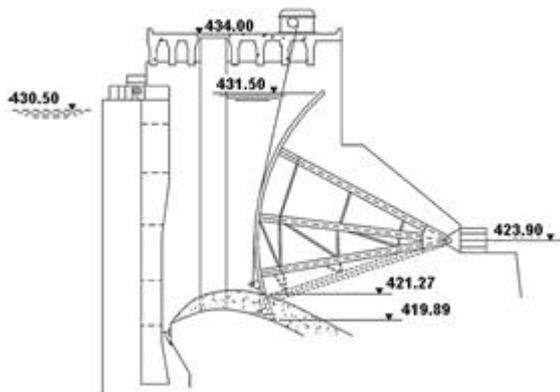


Figure 2-43: Spillway crest heightening at Cachoeira Dourada Dam, Brazil (Erbisti, 2004)

### Inflatable weirs (Figure 2-42)

- Very successfully installed in USA, Japan, Germany, Norway, Australia, France, etc.
- Inflatable weirs is a quick installation
- Low cost
- Sealing is provided directly on the concrete surface by pressure of the rubber weir
- The inflatable weir does not concentrate loads
- They match well with existing structures
- Inflated by air, with a simple low-cost device, of simple maintenance
- It can be equipped with a mechanism to ensure an automatic deflation during flooding

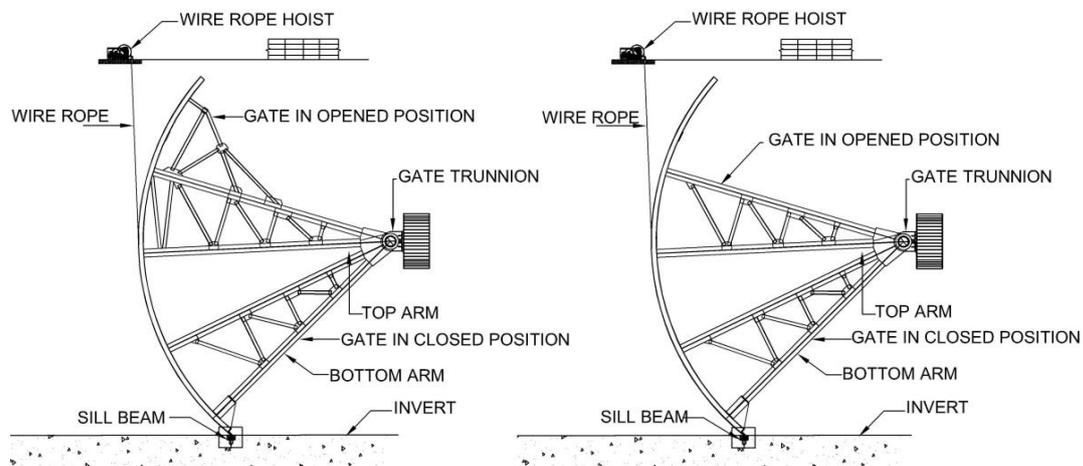


Figure 2-44: Top heightening of Furnas Dam, Brazil.

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## Chapter 3. SPILLWAY

### 3.1 Overview

Spillways are protection structures or safety devices in a dam-reservoir system. The safety of any dam is therefore directly related to the adequacy of its spillway capacity & operational reliability of the its hydraulic and mechanical components. The importance of spillway in management of the dam-reservoir system safety begins with handling the Inflow Design Flood (IDF) for the dam, during the operational life of the reservoir to ensure its serviceability and continued availability during.

Updated hydrological studies, have alerted many dam operators about the need of greater spillway capacity in reservoirs all over the world. According to ICOLD statistics, overtopping is the main cause of dam failures being inadequate capacity of spillway. USACE has reported that almost 30% out of 80,000 dams, within USA, had inadequate spillway capacity. In India, it is estimated that more than 70% of dams do not have spillways with the capacity required to take care of their updated IDF (Fig 3-1). Using as a reference the sample of 198 dam projects in “Dam Rehabilitation and Improvement project (DRIP)”, it was seen that the design flood has increased in 165 (83%) of them.

Apart for the task of reviewing the IDF in Indian dams, there are many deficiencies in existing dams because of aging and deferred maintenance which not only reduces the benefits from these projects but also raises possibility of dam incidents in the future. Also there is increase in hazard potential with time due to increase in population, land use, infrastructure, properties etc. in the downstream areas.

After a dam safety review is done , there is often a need to increase the spillway capacity due to the increase in the design flood to (1)

Present criterion regarding selection of IDF for a particular dam (viz. PMF/SPF/100year flood) as per the latest Indian/International standards. (2) Availability of more data for using in studies and (3) Use of recent and state of the art procedures instead of the earlier empirical formulae. As a result of all that, when using the present IDF to design or to check the spillway capacity, that results in most cases, significantly larger than that used in the original design and larger than actual capacity of the existing spillways.

The present Manual cover very few aspects related to the hydraulic design of spillways; if required, the user may refer to the extensive technical literature and research papers already available on these subjects, especially, in various publications of ICOLD, USBR, USACE, FEMA and others, International Standards and Indian references on the subject as Khatsuria R.M. (2005) and Indian Standards (BIS). As this Manual deals with existing dams, subjects covered here are evaluation of the hydraulic/hydrological safety aspects of existing spillways. Focus is their operation, study of the probable modes of malfunctioning or failure and rehabilitation by repairing and upgrading non-functional structures.

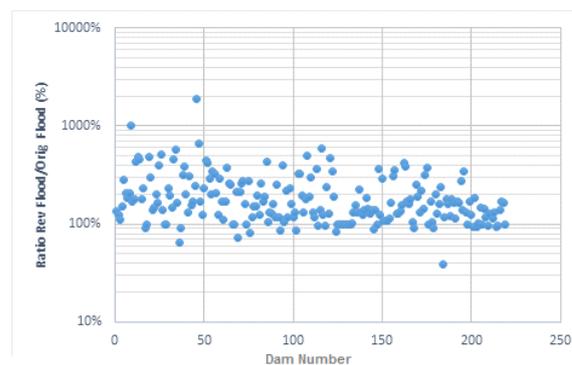


Figure 3-1: Ratio of “Revised Flood/ Original Flood” for reservoirs in India under DRIP

### 3.1.1 Definition and Function

The most common definitions of spillways as per various renowned international organizations are:

**USBR:** “A hydraulic structure that passes normal (operational) and/or flood flows in a manner that protects the structural integrity of the dam and/or dikes”.

**FEMA and USSD:** “A structure over or through which flow is discharged from a reservoir”.

Spillways are required to be hydraulically sized to safely pass floods equal to or less than their IDF.

As explained earlier the spillway is the main protection structure or safety valve for the dam and its reservoir. The focus of hydraulic safety in spillways lies in guaranteeing adequate spillway capacity and satisfactory hydraulic behavior for the entire range of discharges up to its IDF.

### 3.1.2 Classification

There are several ways to classify spillways, the most basic being based on how the spill waters are carried to the downstream river: by surface spillways or by tunnel spillways. Surface spillways are more common than tunnel spillways, leaving the later to dam sites, mainly, where there is not enough space for a surface spillway.

Spillways are internationally classified according to: (1) Type of hydraulic control and (2) Function within the dam-reservoir system.

Based on hydraulic control, spillways are:

- **Controlled:** Releases from reservoir are controlled by operation of a mechanical element (gate). The outflow discharges depend on the number, size of the gates, spillway crest level, geometry and efficiency of the spillway’s control section.

- **Uncontrolled:** Releases from the reservoir depends on the geometry and the efficiency of the control section. Water overflows without any restriction after the reservoir level exceeds the crest level of the ungated control section (FRL).

Another way to group the spillways is by its location in the reservoir and its operation, in two types: (1) On the surface with free discharge or with gates and (2) Submerged with entry to an intermediate level or at the bottom of the reservoir (controlled by gates); Khatsuria (2005) names these bottom outlets as “spillways to discharge floods and sediments”.

Based on its function and provided level of protection, and using USBR definition, spillways are classified as:

- **Service Spillway:** It is a Spillway structure that is normally used for passing flows up to and including the maximum design discharge without any significant damage to the dam, dike, or appurtenant structures. It may be either gated or ungated.
- **Auxiliary Spillway:** It is a structure that is infrequently used and by its use augments the service spillway discharge capacity. During operation the auxiliary spillway may be subject to some degree of damage due to releases up to and including the maximum design discharge.
- **Emergency Spillway:** It is a structure that is designed to provide additional protection against overtopping of a dam and/or dike and is intended for use under unusual or extreme conditions such as erroneous operation or malfunction of the service spillway or outlet works during IDF, larger floods, or remote floods (such as the PMF), or other emergency conditions.

In Indian engineering practice (IS-4410 Part 9, Reaffirmed 2001), these terms are defined

as follows. The terms Auxiliary and Emergency are synonymous.

*Main spillway: “the spillway which is designed to pass the spillway design flood which usually comes into operation. The main spillway may be assisted by the emergency spillway in passing the design flood”.*

*Service spillway: “any spillway that is normally utilized to discharge surplus water. The downstream channel is protected by paving, so that it is not damaged due to the high impact and velocities of the water”.*

*“Emergency spillway: a natural or excavated channel, usually some distance away from a dam provided to permit the release of extraordinary flood flows or flood discharge beyond the capacity of the service spillway. Control gates are seldom furnished and a low embankment of earth may be used to allow the water surface to rise above the crest of emergency spillway. If continued inflow causes overtopping of the embankment plug, it is intended that the plug shall wash away, releasing the excess of water without endangering the main dam. It is also called Auxiliary Spillway”.*

Here it is worth highlighting ICOLD's comments (2007) on these features of the spillways:

- *“The terms “Auxiliary” and “Emergency” for qualifying the function of a spillway are not unanimously understood by the profession. Admitting that the “Service” or “Operational” spillway is the structure (or structures) through which the full Spillway Design Flood hydrograph is discharged, and then any other spillway included in the project could be termed as an auxiliary or emergency spillway”*
- *“The recent practice considers the “design flood” as the flood, which is normally computed, based on probability analyses of hydrologic data, and which must be discharged without impairing the normal operation of the project. On the other hand, the “check flood” or a “Project Design Flood”, which is commonly the PMF or a variation of the concept, is accepted as being the maximum flood event to be supported by the project without incurring a failure of the project such as embankment overtopping”.*

- *“In addition, it might as well consider the case of uncertainty in defining the maximum flood limits and/or of some accidental possible limitation in the discharging capacity of spillway structures. These facts characterize an emergency situation”.*
- *“Based on these concepts, an “Auxiliary Spillway” can be defined as a spillway that complements the capacity of the service spillway to discharge the total Project Design Flood (or Check Flood)”.*
- *“The “Emergency Spillway” could be understood as a supplementary discharge organ that would enter into operation if either the incoming flood, for any reason, is greater than the flood for which the Service and the Auxiliary spillways combined can take care of, or if the normal capacity of these structures is harmed by an unusual event such as gate malfunction, clogging of the spillway passage or emergency cut-off of a powerhouse.*
- *“The use of an Emergency Spillway is, however, a decision that is strictly connected with the possibility of occurrence of floods greater than the Project Design Flood. The Auxiliary Spillway in most cases is used for allowing a reduction of design flood assigned to the Service Spillway, and as a consequence will be operated to take care of the more infrequent floods with higher return periods”.*

In these definitions, it is important to differentiate the use of the terms: IDF: Inflow Design Flood (or SDF: Spillway Design Flood), PDF: Project Design Flood and PMF: Probable Maximum Flood. Khatsuria (2005) mentions that usually the “Safety Check Flood or Project Design Flood” is the PMF. Hydrological aspects about selecting flood are in CWC’s “Guidelines for Selecting and Accommodating Inflow Design Floods for Dams”.

In order to guarantee the operation of the reservoir with one or two spillways (auxiliary or emergency), complementary to the service spillway, an important issue to define is the sequence of their operation and at what

reservoir levels they would start functioning for management of the hydrological risk.

The most frequent project layout envisages a single service spillway only. There are some cases where the service spillway is controlled by gates and in which a second spillway has been included as an emergency (or back up work) to operate in cases of any functional problem occur with the. Use of an auxiliary spillway responds to hydrological uncertainties, but is dependent on availability of suitable sites, and cost of the works. It could be said that hydrological safety is associated with service and auxiliary spillways, and operational safety with emergency spillway, but this is not always so, because emergency spillway is also added to augment the rare flood's release capacity of the reservoir, even when service spillway is a non-controlled type. Figure 3-2 shows a dam with both spillways: service and auxiliary, and an example of emergency spillway.

Another classification of spillway is according to its spillway capacity. But as it is arbitrarily defined it is not used internationally. Some dam's owners and engineering organizations use this type of classification, for example, Table 3-1(USBR, 2014) presents classes according to ranges of spill discharge:-

Location of Reservoirs	Number of Reservoirs	% of reservoirs with respect to 5,745
One State	2,394	41.67 %
Three States	3,931	68.42 %
Six States	4,633	80.64 %
Twenty four States	1,112	19.36 %
All Country – 30 States	5,745	100.00 %

Table 3-2: Geographical distribution of reservoirs at India.

ICOLD's former definition of large dam used as a reference, a spillway's capacity of 2,000 m<sup>3</sup>/s, highlighting the size and

importance of spillways with this capacity or greater. A capacity greater than 20,000 m<sup>3</sup>/s have been named by ICOLD (2016) as a Very Large Spillway, adding another figure of unit discharge greater than 130 m<sup>3</sup>/s/m.

Spillway Capacity (m <sup>3</sup> /s) (*)	Spillway's Classification
Less than 50	Small
50 to 1,000	Modest
1,000 to 1,500	Medium
1,500 to 3,000	Large
Greater than 3,000	Very Large

Table 3-1: Spillway capacity and classification (USBR, 2014)

In India, as per IS11223, the maximum water level now corresponds to all spillway bays/gates as operative. However, the reservoir level is also checked for the contingency of 10% spillway gates (minimum one gate) as inoperative and a reduced but acceptable freeboard is allowed over that.

Table 3-2 shows the Geographical distribution of reservoirs/ dams in India by location, Table 3-3 shows the Distribution of reservoirs/ dams in India as per spillway capacity (state-wise) and Figure 3-3 shows a frequency distribution of spillway's size in India according to range of its capacity, based on the National Register of Large Dams (NRLD).

Within the NRLD, there is a special group of 70 dams called "Dams of National Importance", defined by heights greater than 100 meters and/or reservoir storages greater than 1 km<sup>3</sup>. Table 3-4 presents the spillway's size for these dams of national importance, where more than 80% of the dams have spillway's capacity greater than 3,000 m<sup>3</sup>/s.



Figure 3-2: An illustration of service and auxiliary spillways located in dam and emergency spillway (fuse plug) located far away from the dam in reservoir rim

N°	State listed by number of reservoirs	Number of reservoirs at state	SPILLWAY DESIGN CAPACITY (m <sup>3</sup> /s)							Maximum design capacity (m <sup>3</sup> /s)
			Unknown	< 1,000	1,000 to 3,000	3,000 to 10,000	10,000 to 20,000	20,000 to 60,000	> 60,000	
			0	1	2	3	4	5	6	
1	MAHARASHTRA	2394	108	1870	283	91	26	14	2	67,373
2	MADHYA PRADESH	906	30	826	21	13	7	5	4	106,000
3	GURAJAT	631	6	427	106	63	17	11	1	84,949
4	CHHATTISGARH	258	4	239	7	4	2	2	0	26,708
5	KARNATAKA	232	4	175	25	23	2	3	0	37,945
6	RAJASTHAN	212	2	138	43	19	3	7	0	32,411
7	ODISHA	204	2	167	18	11	3	2	1	66,676
8	TELANGANA	184	4	124	22	16	9	7	2	508,000
9	ANDRA PRADESH	166	26	81	25	21	8	4	1	91,400
10	UTAR PRADESH	130	2	91	18	8	8	2	1	111,328
11	TAMIL NADU	118	4	80	26	6	2	0	0	11,179
12	JHARKHAND	79	5	53	9	5	5	2	0	32,340
13	KERALA	61	10	26	17	7	1	0	0	14,200
14	WEST BENGAL	30	0	24	2	2	2	0	0	15,400
15	BIHAR	26	0	19	3	4	0	0	0	3,818
16	UTTARAKHAND	25	1	5	7	7	5	0	0	19,824
17	HIMACHAL PRADESH	21	0	3	3	10	3	1	1	26,500
18	JAMMU AND KASHMIR	17	1	6	3	4	1	2	0	22,500
19	PUNJAB	16	2	12	0	1	0	1	0	20,678
20	MEGHALAYA	10	2	1	4	2	1	0	0	10,440
21	GOA	5	0	4	1	0	0	0	0	1,450
22	ARUNANCHAL PRADESH	4	0	0	0	4	0	0	0	9,216
23	ASSAM	4	0	1	0	1	1	1	0	37,500
24	MANIPUR	4	0	2	2	0	0	0	0	2,240
25	ANDAMAN AND NICOBAR	2	0	2	0	0	0	0	0	405
26	SIKKIM	2	0	0	1	0	1	0	0	12,500
27	HARYANA	1	0	1	0	0	0	0	0	919
28	MIZORAM	1	0	0	0	1	0	0	0	4,450
29	NAGALAND	1	0	0	0	1	0	0	0	5,977
30	TRIPURA	1	0	1	0	0	0	0	0	481
	<b>TOTAL</b>		213	4378	646	324	107	64	13	
	<b>RANK OF CAPACITY</b>	5745	1	2	3	4	5	6	7	
	<b>%</b>		3.7%	76.2%	11.2%	5.6%	1.9%	1.1%	0.2%	

Table 3-3: Spillways of reservoirs of India - Data from NRLD published by CWC - June 2019

Spillway Capacity (m <sup>3</sup> /s)	Number of dams	Percentage
Unknown	5	7.10 %
< 1,000	4	5.70 %
1,000 to 3,000	4	5.70 %
3,000 to 10,000	18	25.70 %
10,000 to 20,000	18	25.70 %
20,000 to 85,000	21	30.00 %
	70	100.00 %

Table 3-4: Size of Spillways in “Dams of National Importance” (CWC).

Some spillways of Indian dams and in dams of other countries, controlled (gated) and uncontrolled (ungated), along with some

relevant data are shown in Figures 3-4 to 3-10.

### Number of Dams vs. Spillway Capacity

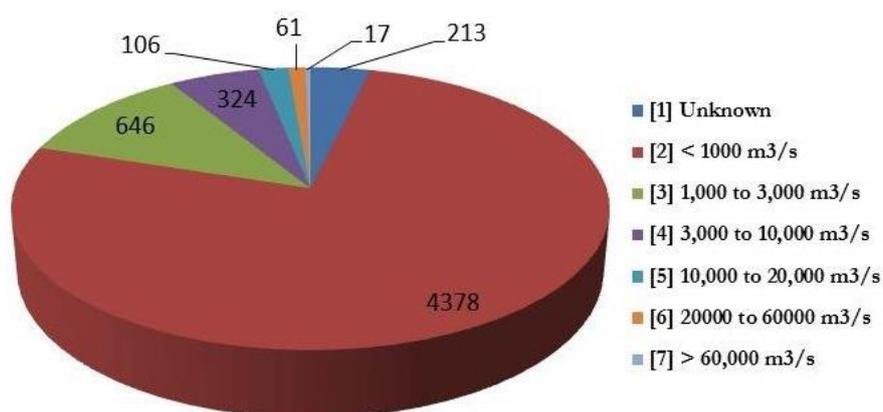


Figure 3-3: Frequency distribution of dams according to spillway capacity in India



Figure 3-4: Sardar Sarovar Dam (Narmada river, Gujarat, India)

Concrete Gravity dam, height: 136.7 m. Service spillway: Controlled, 30 radial gates, Spillway Capacity 85,000 m<sup>3</sup>/s, Reservoir capacity: 9.5 km<sup>3</sup>.

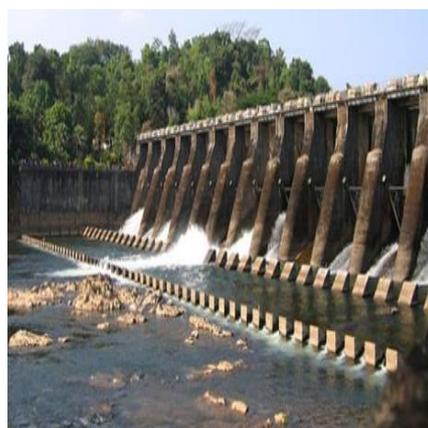


Figure 3-5: Kullar Dam (Valapattanam river, Kerala, India)



Figure 3-6: Mulla Periyar Dam (Chitri river, Kerala, India)  
Masonry Gravity dam, Height: 53.7 meters. Service spillway: Controlled ogee crest; Spillway Capacity 3,455 m<sup>3</sup>/s. Reservoir capacity: 443.23MCM.



Figure 3-7: Chitri Dam (Chitri river, Ajara, India)  
Earth fill dam, Height: 55.1 meters. Service spillway: Un-Controlled; Spillway Capacity: 571 m<sup>3</sup>/s, Reservoir capacity: 53.4MCM.



Figure 3-8: Indira Sagar Dam (Madhya Pradesh)  
Concrete Gravity curved dam, Height: 91.5 meters. Service spillway: Controlled with 21 radial gates; Spillway Capacity 65,670 m<sup>3</sup>/s, Reservoir capacity: 12.2 km<sup>3</sup>



Figure 3-9: Strontia Dam (South Plate river, Colorado, USA)



Figure 3-10: Service spillways. Control structure: free discharge, straight shaped crest. Burdekin Falls Dam; Australia. Spillway Capacity: 64,600 m<sup>3</sup>/s

### 3.1.3 COMPONENTS

A typical spillway for a dam project of any type: storage, run-of-river, and pumped-

storage facilities, has several components from the reservoir to the river channel downstream that receives the discharge. Not all types of spillways have the entire set of

components. Terms, hydraulic and structural criteria and details are presented in IS-4410 Part 9 (Reaffirmed 2001) and IS-5186 (1994).

According to USBR, these components and their functions are:

- Approach or inlet (upstream) channel and safety/debris/log boom: It conveys water from the reservoir to the inlet structure or to the control structure if there is no inlet structure. Also, in some dams there is provision for retention and handling of floating debris (tree trunks, trash, other) to avoid obstruction.
- Inlet structure: It conveys water from the approach channel to the control structure and is intended to improve approach flow conditions to the control structure.
- Control structure: A crest structure or grade control sill, with or without hydro-mechanical elements viz. Gates, Bulkheads or Stop logs along with associated operating equipment like hoists, gantry crane etc. and with other structural elements like piers, bridge, etc.) The hydraulics of the control structure establishes the discharging capacity for the spillway.
- Conveyance features (Chute, conduit, sluice, tunnel or in combination): It conveys water from the control structure to the terminal structure. The conveyance features may include combination of elements such as chutes with both mild and steep slopes, combinations of conduits, tunnels, and chutes.
- Terminal structure (Energy dissipater such as a hydraulic jump stilling basin, flip bucket with or without pre-excavated plunge pool etc.): This structure dissipates most of the kinetic energy associated with

moving water and leads waters to the exit channel.

- Exit channel: Such channels are provided in some dams especially where the spillway is located in one of the flanks and not in the river bed to convey water from the terminal structure to the river or stream in the downstream.
- Measures to control sediment accumulation in reservoir especially near the dam and inlet areas.

The need of these components depend on type of dam, layout, type of spillway, location of the spillway (in the river bed, in an abutment or at any place at reservoir rim), topography (local relief and slope), geology (soil or rocks), function of spillway (service, auxiliary or emergency) and operation needs. The minimum set of spillway's components are two: a control structure and a downstream terminal energy dissipation structure delivering the water, into the main river course.

Table 3-5, adapted from USBR, presents types of spillways with their components, common function and usual upper limits of capacity.

In this chapter, Spillway's components that are presented and analyzed are Control and Conveyances structure. Approach Channel is covered in Chapter 2. "Reservoirs"; and Terminal Structures and Exit Channels are covered in Chapter 5 "Energy Dissipaters".

Both controlled and uncontrolled additional spillways can be considered as per site-specific conditions to increase the spillway capacity in hydrologically unsafe dams.

Some advantages of uncontrolled spillway structures are:

- More reliable and safe since it functions alone without any mechanical element.

- Less likely to be affected by obstruction (floating debris).
- Not dependent on operation - Not vulnerable to malfunction due to human error.
- Less maintenance dependent.
- Currently there are options among uncontrolled spillways (e.g. labyrinth spillway, piano key spillway etc.) that can overcome to some extent the requirement of long length and large space required for uncontrolled spillways.
- Under the DRIP, ungated flush bars and fuse plugs have been provided as additional spillways, to take care of inadequate spillway capacity, in some dams in India.

Some advantages of controlled (gated) spillways are:

- Reduced cost of spillway. Usually its cost is lower than free crest weir.

- Can pass large discharges
- Requires much lesser length and space as compared to uncontrolled spillways

In rehabilitation of spillways, there is no accepted rule for selection of an uncontrolled or controlled spillway.

All possible alternatives need to be examined and the final selection has to be on merits and techno-economic considerations.

Figures from 3-11 to 3-20 shows several types of control and conveyance structures.

As per National Register of large dams (NRLD), there are almost the same proportion of controlled and uncontrolled spillways in India. In general, uncontrolled weirs have a straight alignment and have either an ogee or a broad crest. Controlled spillways use radial gates or vertical lift gates placed on either an ogee's weir or a broad crested glacis

No.	Type	Category			Usual upper limit of capacity (m <sup>3</sup> /s)	Typical conveyance feature		
		Service	Auxiliar	Emergency		Chute	Conduit	Tunnel
<b>A</b>	<b>Uncontrolled discharge</b>							
<b>A.1</b>	<b>According to control structure</b>							
<b>1</b>	<b>Crest of various shapes</b>							
1.1	Straight ogee crest	X	X		>3,000	1	3	2
1.2	Curved ogee crest (fan weir)	X	X	X	>3,000	1	3	2
<b>2</b>	<b>Labyrinth</b>							
2.1	Labyrinth weir, different shapes	X	X		3,000	1		
2.2	Labyrinth Piano Key weir (PKW)	X	X		>3,000	1		
<b>3</b>	<b>Others types of control structures</b>							
3.1	Bathhtub	X	X		1,500		3	2
3.2	Orifice wall	X	X		1,500		3	2
3	Side channel	X	X		1,500			2
4	Drop inlet	X	X					
4.1	Morning Glary (shaft)	X	X		1,500		2	1
4.2	Other	X	X		varies			
<b>A.2</b>	<b>According to conveyance features</b>							
<b>5</b>	<b>Chute spillway</b>							
5.1	Various shaped weirs	X	X	X	>3,000	1		
5.2	Baffled apron	X	X	X	2,500	1		
5.3	Stepped chute	X	X		>3,000	1		
5.4	Grade control sill		X	X	>3,000	1		
<b>B</b>	<b>Controlled discharge</b>							
<b>6</b>	<b>Gated control structure</b>							
6.1	Ogee crest	X	X		>3,000	1		2
6.2	Free over fall	X	X		>3,000			
6.3	Orifice	X	X		>3,000	2	1	3
6.4	Side channel	X	X		3,000	1		2
6.5	Drop inlet	X	X		1,500		2	1
6.6	Tunnel inlet	X	X		3,000			1
6.7	Others	X	X		Varies	1		
<b>C</b>	<b>Fuse weir (collapsible weir)</b>							
7.1	Fuse gate		X	X	3,000	1		
7.2	Fuse plug dike		X	X	1,000 to 3,000	1		
7.3	Concrete fuse plug		X	X	1,500			
<b>8</b>	<b>Siphon</b>	X			750	2	1	
<b>9</b>	<b>Unlined channel (earth or rock)</b>			X	Varies			
<b>10</b>	<b>Overtopping: protected dam section</b>			X	Varies			

**Notes:**

Adapted from USBR (2014)

Capacity according to mentioned ranges. An upper capacity &gt;3,000 means sample use of this type for large discharges.

Numbers in conveyance structures order of preference.

Conveyance features: conduit is a surface structure under the dam, tunnel is an underground structure.

Table 3-5: Spillways: Types, categories, capacities and conveyance features (adapted from USBR)



Figure 3-11: Service spillways. On a concrete gravity dam and left abutment of an embankment dam.



Figure 3-12: Service spillways. Control structure: bathtub or duck bill weir



Figure 3-13: Service spillways. Control structure: free discharge, shaped crest, on concrete gravity dams.



Figure 3-14: Service spillways. Control structure: free discharge, shaped crest, straight frontal and side channels.



Figure 3-15: Service spillways. Control structure: drop inlet (Morning Glory) and special weir



Figure 3-16: Service spillways. Controlled weir with radial gates



Figure 3-17: Service Spillways Chutes



Figure 3-18: Service spillways. Steeped chutes on gravity dam (RCC).



Figure 3-19: Typical shaped crested weir (Ogee) controlled with gates.  
Up: Nagarjuna Sagar Tail Pond (India).  
Down: Guri Dam (Venezuela)

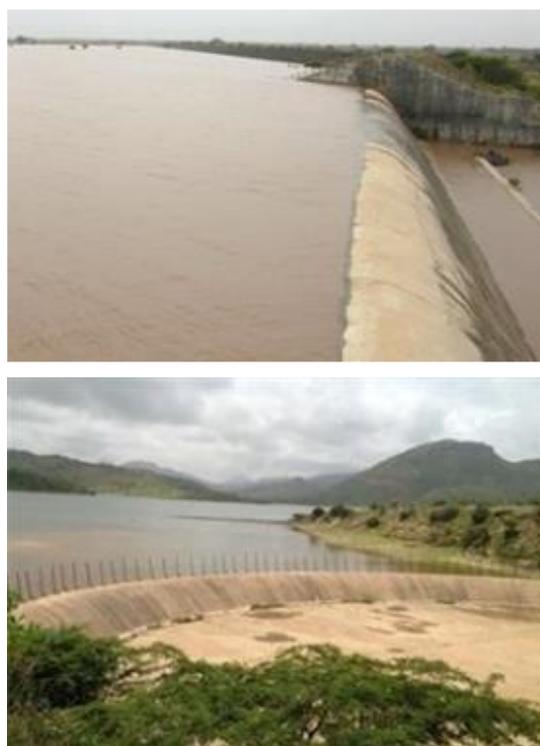


Figure 3-20: Typical free overflow weir (Ogee) with straight and curved axis  
Up: Phophal Dam, Gujarat, India. Earth dam, 30 m. Spillway capacity: 10,580 m<sup>3</sup>/s

Based on available data of Gujarat dams (631 reservoirs) it is seen that 51.5 % spillways are uncontrolled and 48.5 % are controlled, also there is a tendency to threshold discharge of about 2,000 m<sup>3</sup>/s for the uncontrolled spillways, above which the controlled spillways are more dominant (see Figure 3-21).

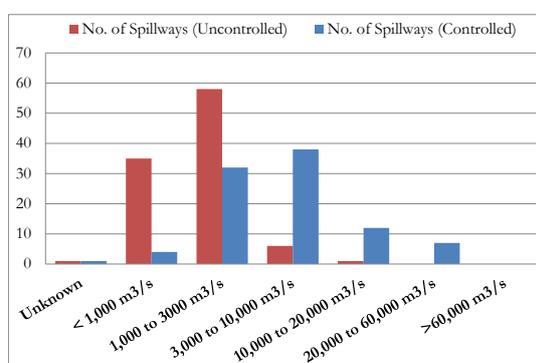


Figure 3-21: Spillways in reservoirs at Gujarat.

## 3.2 Description of Spillways: Control and Conveyance Features

In this chapter only the components of control and conveyance structures are presented. Approach channels and its complementary elements are included in Chapter 2 “Dam and its Reservoir”; Terminal Structures and Exit Channels are covered in Chapter 5 “Energy Dissipators”.

### 3.2.1 Control structures

#### 3.2.1.1 Conventional spillways

##### a) General

Conventional un-controlled spillways generally have classical weir structure; which is normally an overflow control section, with a profile designed with hydrodynamic shape. This weir (or sill and glacis) structure is used

in controlled spillways. The most common crest profiles are Ogee and WES as per USBR/USACE. According to the type of uncontrolled spillway, the weir axis can be straight, curved or circular, and its layout, in relation to flow's direction from the reservoir or approach channel, can be: frontal, lateral and radial. For controlled spillways, weirs or glacis are, commonly, straight and frontal.

For ungated surface spillway, the crest of the weir matches the full reservoir level (FRL); for gated surface spillway, glacis crest is under FRL (to a depth almost equal to gate's height).

The types of spillways with conventional weirs and its conveyance features are:

- **Uncontrolled spillways:** Free surface flow.  
 Conveyance: Glacis of a conventional spillway, discharge carrier of a chute/side channel spillway, shaft/tunnel of a Morning Glory spillway etc.
- **Controlled spillways:** Both orifice flow and free surface flow, that is with gates partially or fully open.  
 Conveyance: Glacis of a conventional sluice spillway, discharge carrier of a chute/side channel spillway, Shaft/Tunnel of a tunnel spillway.

Various types of spillways provided in Indian dams and for dams under DRIP are shown in Figure 3-22. More than 80 to 85% of spillways are of conventional type. The use of Sluice (bottom) spillways has increased recently especially in the Himalayan region where silt load is large. Other types includes some of the non-conventional weirs. Spillways with underground conveyance (tunnel) such as Morning Glory or tunnel inlet (shaped crest weir with/without gates) are not very common. However, a recent reservoir with shaft spillways in India is Tehri dam, which has a combination of five number spillways: Controlled Ogee crest with chute, 2 no, Drop Inlet with Morning Glory

crest and 2 no. Tunnel inlet with shaped crest.(See Figure 3-23).

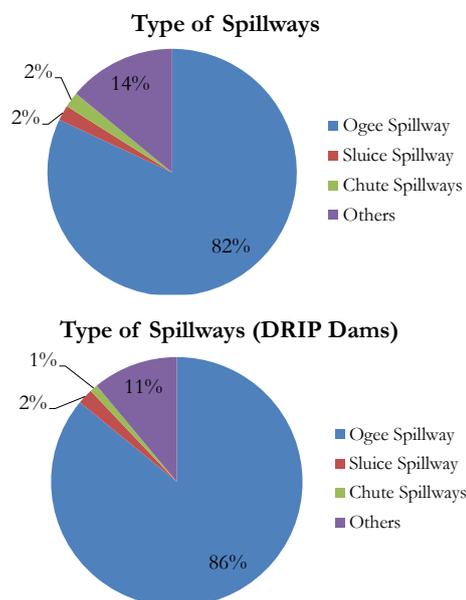


Figure 3-22: Type of spillways in dams/reservoirs of India and for dams under DRIP.

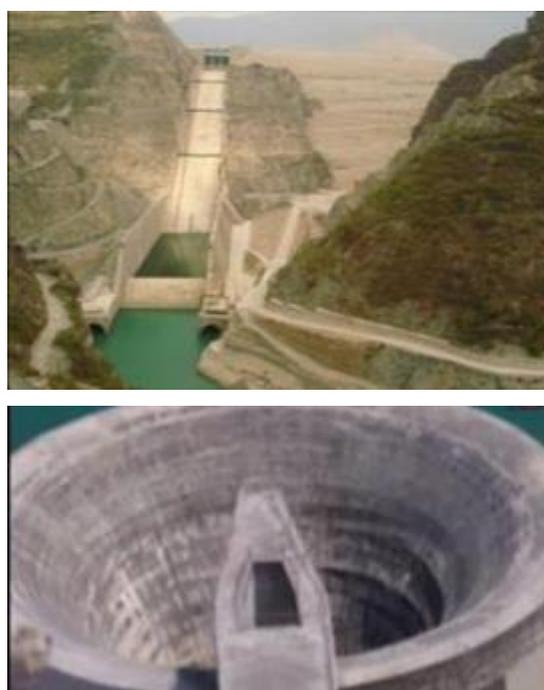


Figure 3-23: Tehri Dam. Chute Spillway & Morning Glory Spillways

In the CWC's sample of "National Importance Dams" (70 dams), most of spillways are Ogee crest type (controlled and uncontrolled). It is seen that controlled spillways are predominant. It is to be mentioned that in this group 2/3 are concrete gravity dams and 1/3 are embankment dams (earth or rock fill). These embankment dams include a concrete gravity section for the spillway. Centrally located spillways in river bed are predominant in concrete gravity dams. Another aspect to mention is the common use of radial gates (more than 75%) over other types of gates.

b) Discharge computations for conventional weirs

First task of an evaluation of hydraulic safety of spillway is to establish its capacity. The rating curve of the spillway is the basic tool to define its capacity, it varies for uncontrolled and controlled weirs, as mentioned previously: free overflow or orifice flow. The equations, letter symbols and terminology used to define the Spillway's Rating Curves correspond to IS-6934 (Reaffirmed 2003), also to Khatsuria (2005), with dimensions in International System.

The equations of both cases are:

For free overflow weir:

$$Q = \frac{2}{3} \cdot C \cdot \sqrt{2g} \cdot L \cdot H^{3/2}$$

or

$$Q = C_d \cdot L \cdot H^{3/2}$$

For orifice flow:

$$Q = C_0 \cdot A \cdot \sqrt{2g \cdot H_0}$$

or for gates:

$$Q = C_g \cdot G_0 \cdot L \cdot \sqrt{2g \cdot H_c}$$

where:

Q = Discharge (m<sup>3</sup>/s) over the weir or through the orifice.

C = Coefficient of discharge (non-dimensional)

C<sub>d</sub> = Coefficient of discharge of weir,  $C_d = \frac{2}{3} C \cdot \sqrt{2g}$  (dimensional), (m<sup>1/2</sup>/s).

C<sub>g</sub> = Coefficient of discharge for flow under de gate (non-dimensional).

L' = Net length of the spillway (m) i.e. clear length between piers.

L = Effective length (m) = L' - (Effects of flow contractions due to piers and abutments).

H = Hydraulic head over the crest of the weir (m) = Reservoir level - Spillway crest level.

H<sub>c</sub> = Hydraulic head (m) above the center line of gate opening = Reservoir level - Centerline elevation of gate's opening.

H<sub>o</sub> = Hydraulic head (m) above the center line of orifice = Reservoir level - Centerline elevation of opening.

H<sub>d</sub> = Design hydraulic head (m)

C<sub>o</sub> = Coefficient of discharge of orifice (non-dimensional)

A = Area of orifice opening (m<sup>2</sup>) considering minimum distance between spillway crest profile and bottom of the gate.

G<sub>o</sub> = Gate opening (m) i.e. minimum distance from weir's surface to gate lip.

g = Acceleration due to gravity (m/s<sup>2</sup>)

For overflow weir, the discharge coefficient depends on the approach flow conditions (reservoir or approach channel), height of spillway crest measured from the channel/river bed level (P), design hydraulic head (H<sub>d</sub>), slope of u/s face, d/s submergence and interference of the d/s apron, radius of circular weir (R<sub>s</sub>).

For gates, the discharge coefficient depends on the type of gate (vertical or radial), inlet shape (ogee, flat, other), crest arrangement (location of gate's seal, at the crest or down-

stream), head ( $H_c$ ), gate opening and approach and downstream flow conditions.

spillway under both uncontrolled and controlled conditions.

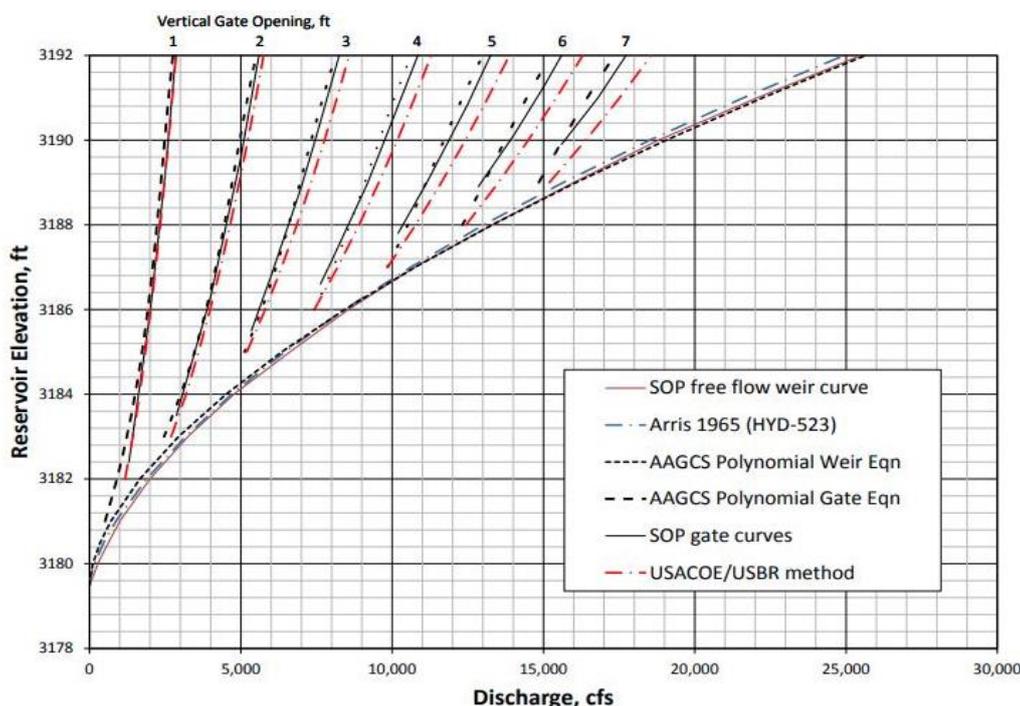


Figure 3-24: Typical Discharge Curves for Free over flow and under partial gate operation i.e. orifice flow condition (USBR, Yellow Tail Dam)

The rating curve ( $Q$  vs.  $H$ ) is unique for the control section, so it defines hydraulic functioning of the spillway. For each type of free overflow shaped weir: frontal, lateral (side channel), shaft (Morning Glory), Duck Bill, and others, parameters “ $C_d$ ” and “ $L$ ” are properly selected from characteristic curves and following design criteria presented in technical references (IS-6934 and Khatsuria, 2005). In the particular case of shaft spillway, its functioning is usually as free overflow but control section can switch from the crest of the weir to downstream conveyance structure (shaft or tunnel), for large discharges ( $Q$ ) and heads ( $H$ ), so flow becomes orifice or pressure type with different rating curve.

For weir or glacis, an ogee or WES shape are preferable as coefficient of discharge are greater than that for broad crested spillway.

Table 3-6 presents the discharge coefficients for free overflow at several weirs to be used for initial evaluation of spill capacity. Figure 3-24 shows typical rating curves for a gated

Free Overflow: Ogee		Shaft Spillway: Circular Weir				
$P/H_d$	C	$H_d/R_s$	$P/R_s = 2$	C	$P/R_s = .3$	C
0.2	1.90	0.2	3.90	2.15	3.98	2.20
0.5	2.10	0.4	3.47	1.92	3.70	2.04
0.7	2.13	0.6	3.10	1.71	3.31	1.83
1.2	2.15	0.8	2.47	1.36	2.58	1.48
3.0	2.18	1.0	2.04	1.13	2.20	1.21
Other Weirs		Controlled spillway radial gates, partially open				
Broad crest weir	1.70	Gate seat at weir crest	$0.67 \leq C_d \leq 0.72$			
Overflow concrete dam	2.20	Gate seat at downstream	$0.68 \leq C_d \leq 0.73$			

Note: “P” – Height of spillway, in this case with vertical upstream face

Table 3-6: Spillways: Discharge coefficients for free overflow and orifice flow (radial gates)

### 3.2.1.2 Orifice Spillways/Breast wall spillways/Bottom Outlets

#### a) General

Orifice spillways, Bottom outlets and Breast wall spillways fall under the category of those spillway structures that can evacuate both flood water as well as sediment from the reservoir. These have become very popular in India especially in Himalayan region where the silt loads are very high and the low spillway crest and large size gates enables a good sediment management, particularly for run of the river Hydro-Electric projects with diurnal storage.



Figure 3-25: Wilson Dam, Alabama. Orifice spillway; flow is controlled by 58 gates (USA, 1926).

In some projects the bottom outlet / orifice spillway has been built in addition to the surface spillways, operating as auxiliary spillway.

b) Orifice spillways

Orifice spillways can operate under heads larger than that in surface spillways resulting in a most economic structure to safely pass the design flood (Wei, 1993). Orifice spillway permits setting of its crest at a significantly low elevation even in a very high dam. A relatively smaller size of radial gates results (compared with a surface spillway). Large heads at the orifice, permits to place the hydro-power intakes at an elevation so as to protect the intakes from both vortex formation. The low crest level of the orifice spillways allows a better sediment management in the reservoir and to safeguard the power intake against entry of sediment. Periodic sluicing and flushing of the sediment is required to be carried out. (Figures 3-25 and 3-26). As a part of the orifice a stop-log slot is provided for maintenance or emergency at radial gate.

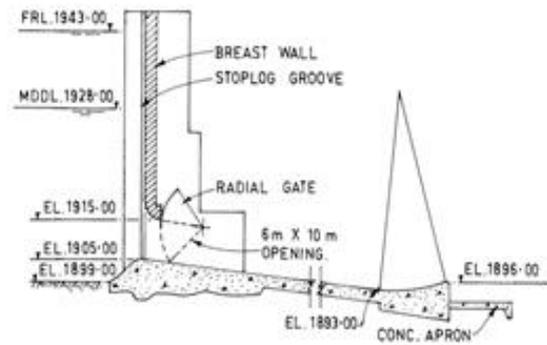


Figure 3-26: Orifice Spillway with radial gates (Khatsuria, 2005)

c) Bottom Outlet

This type of spillway consists in a low or middle level bottom outlet, which is especially convenient for high dams and large volume of flood, in basins with significant sediment yield. Thus, with smaller gates than those used for surface spillways, bottom outlets are a better choice to discharge water and sediments. They have been built, with or without a surface spillway; and can operate separately or simultaneously. Bottom outlets can be located in the body of overflow dam section or at non-overflow dam section. Figures 3-27 and 3-28 are examples of bottom outlets.

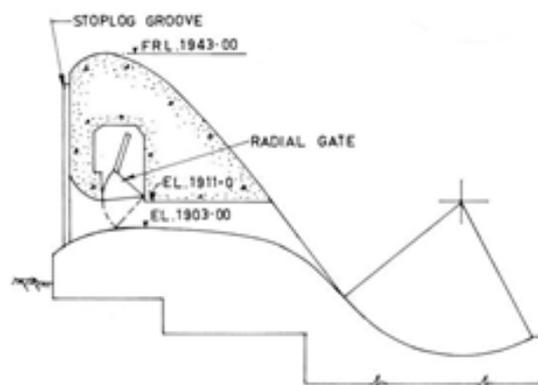


Figure 3-27: Bottom outlet in concrete dam (Khatsuria, 2005)



Figure 3-28: Typical bottom outlet in overflow dam section

#### d) Breast Wall spillway

Breast wall spillway is derived from the flow through a rectangular orifice (whose length is much larger than the vertical dimension). The geometry of the orifice influences its performance and modifies the discharge. There are two types of intake design, Profile “A” where upstream face of the breast wall is in line with the upstream vertical face of the spillway, and Profile “B”, the breast wall has been shifted downstream. Although Profile “B” is hydraulically less efficient, it is the commonly accepted design since it allows an ease installation of stop-log which adds security to the spillway.

Roof profile of rectangle orifice conforms to part of quarter of an ellipse gives better results. Bottom surface, is a profile integrated by an ellipse or flat surface (upstream part) and a parabola (downstream part). This ensures that the spillway floor fully supports the jet and reduces the possibility of cavitation at partial gate openings. Other important detail in its hydraulic functioning is the inclination of the orifice barrel. Figure 3-29 shows typical geometry of orifice (see IS 6934 - Reaffirmed 2003 and Khatsuria, 2005).

The spillway operates as a free overflow spillway for low discharges and as orifice flow for high discharges; in this case, with a

minimum submergence over the crest of 1.7D.

#### e) Discharge computations for orifice and breast wall type spillways

The discharge is given by:

$$Q = C_b \cdot N \cdot A \sqrt{2g \cdot H_c}$$

where,

$C_b$  = Coefficient of discharge of breast orifice, according to orifice shape and profile. Usually  $0.72 \leq C_b \leq 0.90$

N = Number of spans

A = Area of orifice opening = L .D

L = Width of orifice (m)

D = Height of orifice (m)

$H_c$  = Head measured from center line of the gate opening to reservoir level(m) =  $(H - \frac{1}{2} D)$

H = Head measured at weir’s crest (m).

Orifice spillways/Bottom outlets/Breast Wall spillway do not follow a standardized design as conventional spillways, so commonly models are used. They have been used for variable setting conditions. Table 3-7 and 3-8 show data from spillways around the world and from India. From these tables it is seen that these spillways have been in operation successfully for: Heads over the crest: 20-60 m, Discharges: up to 44,000 m<sup>3</sup>/s, Flow per bay: up to 3200 m<sup>3</sup>/s, Width of opening: 5 to 15 m, Height of opening:

4 to 14 m, Discharge intensity: 70-250 m<sup>3</sup>/s/m. Figure 3-30 to 3-35 show examples of this type of spillway

SEE APPENDIX D

Where the physical model of the orifice spillway of the Rattle Hydroelectric Project is analyzed.

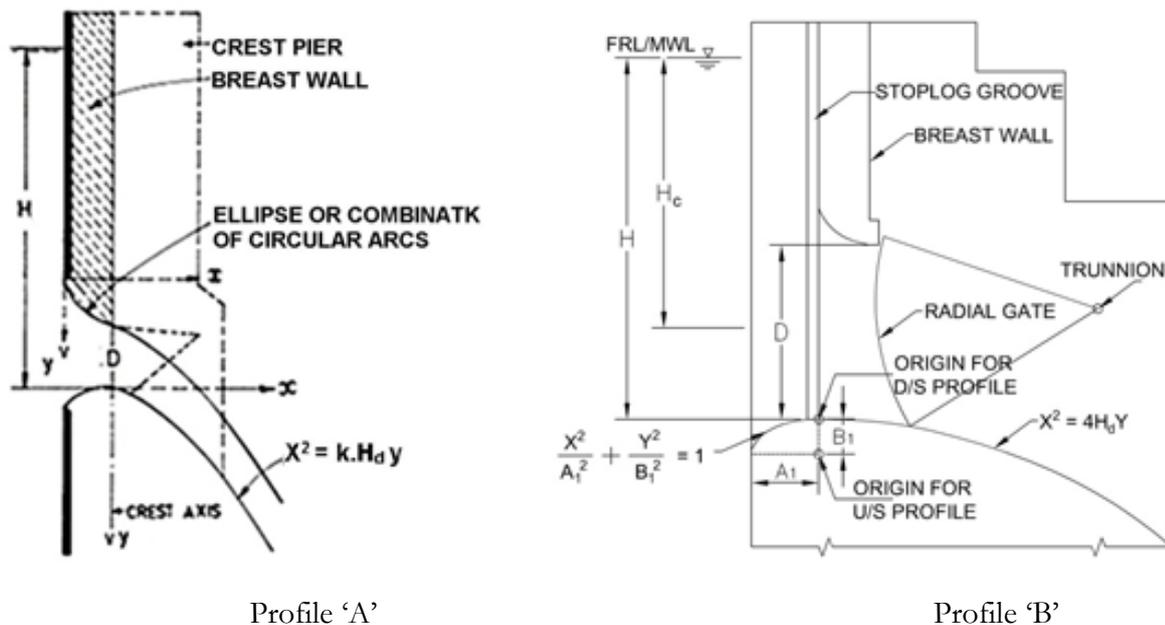


Figure 3-29: Breast spillway. Typical inlet profiles (Khatsuria, 2005)

Project	Country	N <sup>o</sup> of Gates	Dimension of Gates		Total discharge (m <sup>3</sup> /s)	Discharge per bay (m <sup>3</sup> /s)	Discharge Intensity (m <sup>3</sup> /s/m)
			Height	Width			
Clyde	New Zealand	1	9	6	1400	1400	233.3
Jeeba	Nigeria	6	9.5	12	13600	2267	188.9
Magat	Philippines	2	6.0	12.5	3100	1550	124
Roseires	Sudan	5	6.0	11.5	7400	1480	128.7
Chira piura	Peru	3	9.8	12	5500	1833	152.8
Feistritz	Austria	3	5.2	15	3100	1033	68.9
Jupia	Brazil	37	7.6	10.0	44400	1200	120
Sobradinho	Brazil	12	7.5	9.8	22855	1905	194.4
Promissao	Brazil	5	8.6	9.0	6500	1300	144.4
Moxoto	Brazil	20	8.3	10.0	28000	1400	140
Kashm el ghirba	Sudan	7	7.0	7.5	8700	1273	169.7
Mangla	Pakistan	9	12.2	11.0	28600	3178	288.9

Table 3-7: Some large projects with orifice Outlet spillway (Castro, C., 1992)

Sl. N <sup>o</sup>	Project	Hd (m)	Spillway Opening			Design Discharge (m <sup>3</sup> /s)	Specific Discharge (m <sup>3</sup> /s/m)	Cd
			N <sup>o</sup> of bays(n)	W (m)	D (m)			
1.	Chamera I	30	8	10	12.8	20376	254.7	0.84
2.	Chamera III	37	3	12.5	16.5	11400	304	0.78
3.	Dhauliganga	38	2	6	10	2560	213.3	0.8
4.	Kurichu	28	5	10.5	14	12200	232.4	0.83
5.	Nathpa Jhakri	37.5	5	7.5	8.5	5660	150.9	0.88
6.	Nimobazgo	28	5	7	9	4500	128.6	0.84
7.	Pandoh	21.6	5	12	13.5	9939	165.7	0.73
8.	Parbati -II	33	3	6	9	1850	102.8	0.77
9.	Parbati -III	32	2	7.2	14	3300	229.2	0.74
10.	Sewa-II	29.5	4	7	10.8	4020	143.6	0.76
11.	Subhansiri Lower	60	9	11.5	14	3,500	338.2	0.8
12.	Tala	43	5	6.5	13.2	10,490	322.8	0.89
13.	Teesta-V	24	4	9	11.4	4,850	134.7	0.81
14.	Uri-II	24	4	9	11.4	4,850	134.7	0.81
15.	Myntdu	30.5	7	8	12	10,440	186.4	0.78
16.	Pare	29.2	3	10.4	14	5,000	160.3	0.78
17.	Punatsangchhu-II	46	7	8	13.2	16,023	286.1	0.82
18.	Devasari	29	5	8.5	12.5	6,969	163.9	0.77
19.	Mangdechhu	45	4	10	16	8,500	212.5	0.72

Table 3-8: Details of orifice Spillways in India and Bhutan (adapted from Khatsuria, 2005)


 Figure 3-30: Roseires dam, Sudan. Gates 6m (H), 11.5 m (W), Q = 7,400 m<sup>3</sup>/s (SMEC)

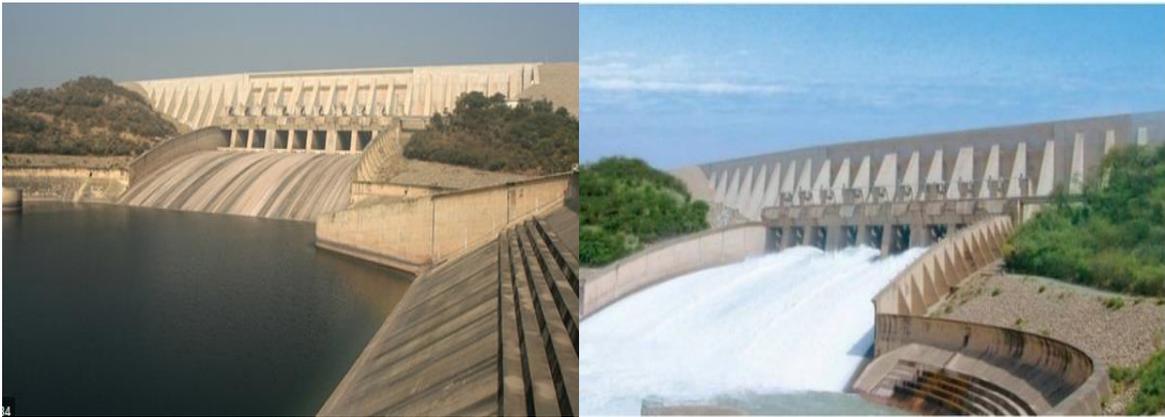


Figure 3-31: Mangla Spillway, Pakistan.  $Q = 28600 \text{ m}^3/\text{s}$ ,  $Q \text{ per bay} = 3178 \text{ m}^3/\text{s}/\text{m}$  orifice spillway, Indus River, Pakistan.  $Q = 28,600 \text{ m}^3/\text{s}$



Figure 3-32: Ranaganadi dam, India.  $H = 24.3\text{m}$ ,  $q = 152 \text{ m}^3/\text{s}/\text{m}$ .



Figure 3-33: Sobradinho Hydroelectric Project Dam, Brazil, 12 gates 7.5m (H)x 9.8m (W). Heat over sill=30.2m,  $Q = 22,855 \text{ m}^3/\text{s}$ ,  $Q_{1\text{bay}} = 1,905 \text{ m}^3/\text{s}$  (Chefs)



Figure 3-34: Jupia Hydroelectric Project, Brazil. 37 gates 6.6m (H)x 10m (W). Heat over sill=17m.  $Q = 44,400 \text{ m}^3/\text{s}$  (CESP).

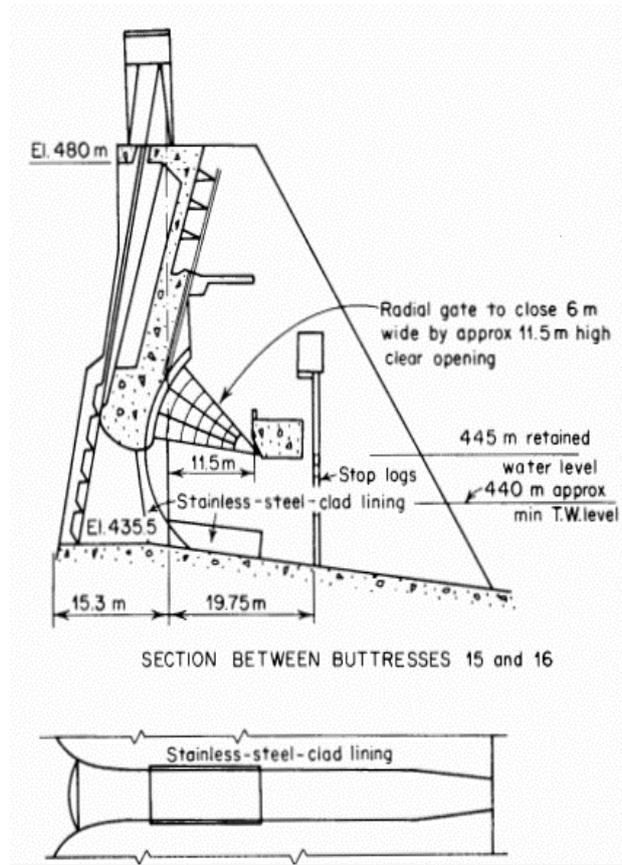


Figure 3-35: Rosaries Orifice Spillway, Sudan, (CY Wei, 1993)

f) Advantages of Orifice Outlet/Spillway

- Better sediment management especially where silt load in the river is high.
- Reduction of height of the spillway gates.
- Reduction of number of spillway bays.
- Ease of regulating flood discharge and water storage.
- Reduction of cost of gates and operating mechanism.
- Permits the dual function of passing floods and sediment.
- A short crest and a high concentration flow, creates optimal flow conditions for sediment flushing from the reservoir.

- The bottom outlet can be built in the first stage of river diversion, and be operative for the second stage, and after the river final closure start operation as a service spillway.

- It can work as outlet for lowering the reservoir after an emergency or a seismic event.

g) Orifice Spillways/Breast wall Spillways/Bottom Outlets: Lessons learned

- Flow concentrations are high; up to 3,178 m<sup>3</sup>/s per bay at Mangla Spillway in Pakistan. Such large concentration of flow may create a challenge for the design of the energy dissipater which may result in a large and costly structure.
- Either gate - radial or slide - can be used for this type of spillway. As the gates work at a head higher than that of normal surface gates they will be heavier and more expensive.
- Operation of Orifice Spillways/Breast wall Spillways/Bottom outlets may bring multiple benefits in the project operation such as better sediment management (flushing), control the reservoir levels during the critical period of first filling, for rapid drawdown in case of emergency, ecological flow, etc.
- During construction, diversion flood can be easily passed through the bottom outlet whose main function is to work as a spillway later.
- The world experience of orifice spillway, breast wall spillway and bottom outlets has been good, in general. Usual problems as experienced in other structures, surface spillways, are similar in orifice spillways as the flow velocity increases.
- World experience reports problems such as:

- a. In the Intake and Gate: Vortex formation in the reservoir, gate vibrations as resulting from partial submergence of gates, pressures pulsations and cavitation. These problems can be solved by a better layout, use of anti-vortex devices to break vortex circulation, adequate operation rules of the gates, floating and fixed racks in the reservoir surface to break vortex formation, etc.
  - b. In the chute: cavitation and abrasion are major issues. Cavitation damages can be avoided by forced aeration. Abrasion, by passing sediments or rock elements from the reservoir is another issue. This problem can be solved by use of special high strength concrete, steel lining should be considered, etc. Particular cases of such spillways discharging at very flat slope may pose a challenge to aerate the flow because of difficulty in controlling the mass of water from the tailrace to return and fill the air cavity.
- In the plunge pool: the scour caused by the spillway flow is a concern especially that near the spillway structure. Pre-excavated plunge pools along with concrete aprons adjacent to the flip buckets are becoming popular. For an important spillway structure, a maintenance gate should be included. This gate, located upstream of the control structure should be preferably designed to close with flow. For maintenance/repairing the energy dissipater that is normally totally or partially submerged, stop logs may be used to isolate the structure from the downstream river. That results in costly training walls.
  - Because of partial or total submergence of the structure, adverse topography downstream, can produce circulation of flows and movements of rock material with severe effect of abrasion on the concrete surfaces. In most cases the solution is use of steel lining or high strength concrete, besides improving the downstream topography, removal of loose stones/boulders what makes complicated the maintenance works.
  - Cavitation damage has occurred in some installations. Flow aeration should be included in the prototype structure, as necessary.
  - Submergence on the downstream is required to be suitably considered in the hydraulic design.
  - This type of spillway can be used as a service spillway or auxiliary spillway or even as an emergency spillway.
  - In this type of spillways, the use of physical models is fundamental to reach best hydraulic performance. Figure 3-36 shows research on model of a large breast spillway for Caruachi Dam (Venezuela). Appendix D presents information about the use of hydraulic models. Figure 3-37 presents pressure measurement in a model.



Figure 3-36: Breast spillway model Esc 1:27, as tested as an alternative of service spillways for Caruachi Dam, Venezuela,  $H=50\text{m}$ ,  $Q=30,000\text{ m}^3/\text{s}$  (EDELCA)

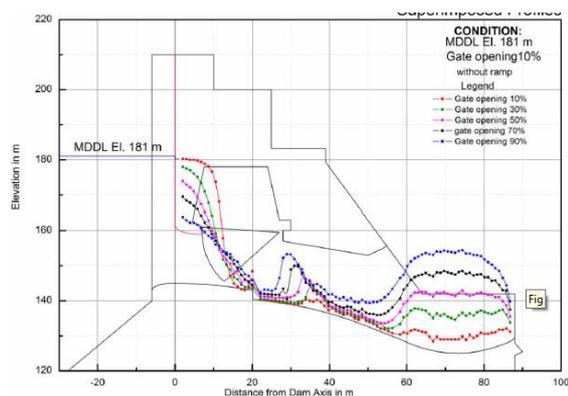


Figure 3-37: Model of orifice spillway, pressure measurement (Chavan,B.S.;2019)

SEE APPENDIX B.1

Where the Caruachi Project spillway is analyzed; the 2 purposes for which it was designed, the models made and its final behavior.

#### 4.2.1.3 Non-conventional spillways

##### a) General

Non-conventional spillways are of several types, some of them of recent development (about 10 years old), that have been used both for the design of new dams and rehabilitation of existing dams. As a result of world-wide concern about dam’s safety and to avoid overtopping due to limited capacity of spillways, these weirs are also being used for upgrading the spillway capacity of existing old structures.

Among these weirs, there are two approaches of functioning:

- A structure that will be robust against hydraulic forces imposed during IDF conditions, as conventional weir.
- A structure that will collapse when a threshold hydraulic load is reached, which increases flow area and discharge.

Usually, the second option envisages emergency spillways that function as a fuse plug which could be accepted, provided their

functioning is guaranteed and consequences downstream allow this discharge.

In relation to spillways and their classes: service, auxiliary and emergency, it is important to mention that engineering criteria for defining and designing them could vary according to expected function, security, consequences of its operation and cost (this last aspect is related to expected level of risk). Some organizations use the concept “robustness” associated with the function of these works (that is “the quality or condition of being strong and in good condition or unlikely to break or fail”).

Apart from the criteria for comparing service, auxiliary and emergency spillways presented in 3.1.2. at the beginning of this chapter, USBR adds the above concepts to definition of these three spillways:

- “Service spillways are typically very robust, erosion-resistant structures consisting of mostly cast-in-place reinforced concrete and riprap channel protection”.
- “Auxiliary spillways may be less robust, erosion-resistant structures consisting of some cast-in-place reinforced concrete, riprap channel protection and/or unarmored excavated channels; some degree of structural damage and/or erosion may be expected due to releases up to and including the maximum design discharge”.
- “Emergency spillways are the least robust, erosion-resistant structures consisting of some cast-in-place reinforced concrete, riprap channel protection, and/or unarmored excavated channels”.

Different types of non-conventional spillways include the following:

- Fuse plug (erodible embankment or dyke)
- Labyrinth weir
- Piano Keys weir (PKW)
- Fuse gates
- Fuse plug (concrete blocks)
- Overtopping of dam with protected section

- Unlined channels

This group of non-conventional spillways have been used internationally for rehabilitation of existing spillways; some of them had been used in DRIP project. They are presented in the order of “more time in use” to “more recent use”. Aspects such as hydraulic efficiency, security, easiness to function, cost, construction etc. are required to be considered during the rehabilitation stage .

b) Fuse plug (erodible embankment)

“Fuse plug, or a breaching section, is an erodible predetermined separate section of an earth dam designed to wash out when the inflow is in excess of the spillway capacity and the reservoir behind it reaches a specified level” (Khatsuria, 2005). In existing reservoirs, a fuse plug is an appurtenant work added when the project spillway does not have sufficient capacity to pass IDF; in particular, when modifications of service spillway structures are not an option but space, in dam site or along the reservoir rim, is available.

The fuse plug is an embankment with limited height built mainly with an engineered earthen section of selected soils: a core of cohesive impermeable soil and non-cohesive shell material. This embankment is designed to fail sacrificially in order to prevent a more catastrophic failure of dam. During overflowing, shell material is eroded away, the core fails as a cantilevered structural element, leading to rapid, reliable breach initiation. For this functioning the best location of core is with upstream inclination.

A fuse plug behaves as a broad crested weir according to relation between energy of flow on the crest ( $H$ ) and its length ( $J$ ) ( $0.08 < H/J < 0.5$ ) as shown in Figure 3-38. Its capacity to discharge can be defined during the wash out processes. For controlling overflow and wash out process, a pilot channel is provided in a short length of the fuse plug with crest slightly lower than the

rest of the remaining fuse plug and consisting of materials of highly erodible nature, and designed to fail first.

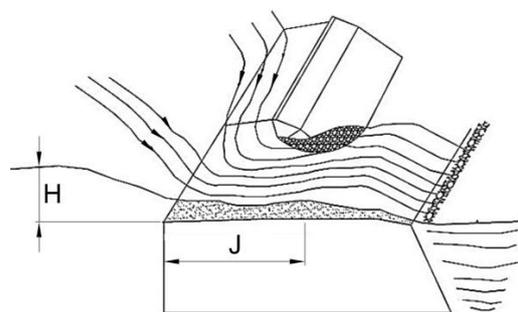


Figure 3-38: Flow through breached portion of a fuse plug

Equation for discharge over the fuse plug (in SI units), follows:

$$Q = C * L * H^{3/2}$$

$Q$  = Discharge ( $m^3/s$ )

$L$  = Length of weir perpendicular to flow (m)

$C$  = Discharge coefficient ( $m^{1/2}/s$ ). It varies during failure process

$H$  = Depth of flow (m). It varies during failure process, from head over the crest to head through the breach.

Based on model studies conducted by USBR, the recommended values of coefficient of discharge ( $C$ ) are as under:

- During washout in one direction: 1.51
- During washout in both directions: 1.71
- After washout is complete: 1.44

Figure 3-39 shows a typical fuse plug section. Figure 3-40 shows Keepit Dam (55 m high), built during 1940-1960 in New South Wales, Australia. Rehabilitation of the dam was carried out during 2010-2011 by upgrading the reservoir works to comply with safety standards: increasing the dam height, raising the crest level with a parapet, change of the radial gates, adding two low saddle dams and two fuse plug spillways as shown. Figure 3-41 shows fuse plug of Timah Tasoh Dam, Malaysia.

Table 3-9 presents a summary of aspects related to use, design, construction and control of fuse plug of the type - erodible earth dam. Figure 3-42 depicts typical mode of failure of a fuse plug during operation.

- Some examples of fuse plug projects (fusible embankments) include :

a) Fuse plug considered in original dam's design

Strontia Springs dam, USBR; Colorado; USA. Dam height: 91 m, built in 1979-1982. Auxiliary spillway: fuse plug (earth dike), for discharges exceeding 340 m<sup>3</sup>/s (with a range of discharges of 339.8 upto 991.1 m<sup>3</sup>/s). Total combined capacity (service + auxiliary spillways): 2,579 m<sup>3</sup>/s. Location of fuse plug: on left abutment discharging on natural rock slopes. (see Figures 3-43 to 3-45).

b) Fuse plug included during rehabilitation of Bartlett dam; USBR; Arizona, USA. Concrete multiple arch buttress dam, built 1936-1939. Dam height: 87.5 m.

Reservoir capacity: 217 hm<sup>3</sup>. Auxiliary spillway: Fuse-plug (earth dike) with intermediate walls, for design discharge of 7425 m<sup>3</sup>/s. Location: On left abutment discharging on natural rocky slope. During operation of fuse plug (PMF), maximum water elevation in reservoir increases leaving a 1 m freeboard. (See Figure 3-46).

- Case of fuse plug's functioning:

Silver Lake Dam, Dead river, Michigan, USA. This is an old embankment dam (10 meters high) rebuilt in 1944. Its emergency spillway (fuse plug) was breached on May 2003. This failure emptied almost the whole reservoir (36 hm<sup>3</sup>) and the water flooded downstream zones and overtopped the city of Marquette's Tourist Park dam. No lives were lost and no major injuries occurred since an available emergency action plan was activated. The fuse plug was rebuilt in 2008, and the reservoir is operative (see Figure 3-47).

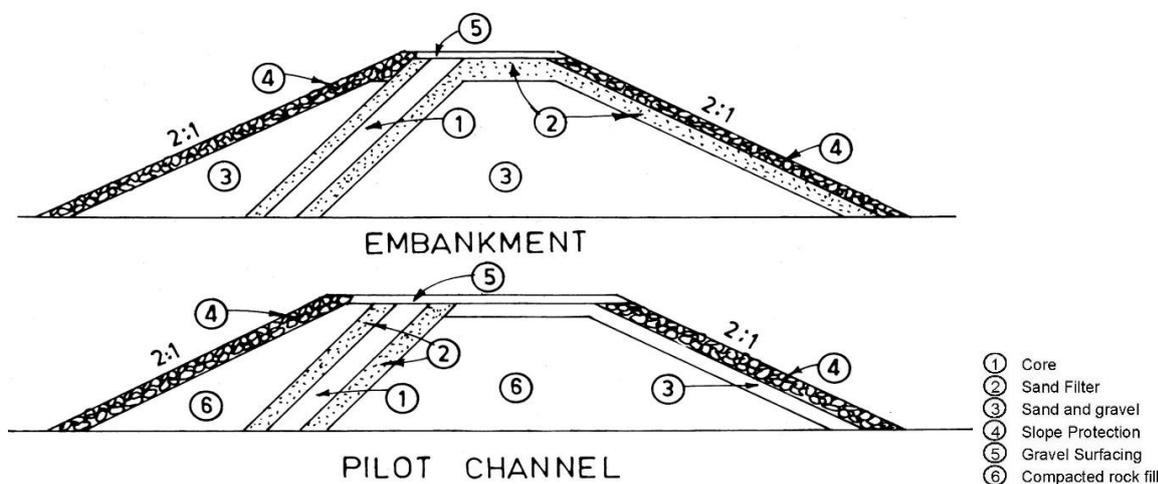


Figure 3-39: Typical fuse plug's section

<p><b>Plan view and arrangement of fuse plug (Erodible Embankment)</b></p>	<p>Earth dam or dike with rectilinear or curved axis, with limited height.</p>	
<p><b>Use</b></p>	<ul style="list-style-type: none"> <li>• Auxiliary spillway</li> <li>• Emergency spillway</li> <li>• Typical use when service spillway cannot be modified, and there is a place along</li> </ul>	

	<p>the reservoir for this additional work.</p> <ul style="list-style-type: none"> <li>• For any type of dam: Concrete, Rock fill, Earth fill or Combination</li> <li>• As a fuse plug, it operates once during dam life time, then it has to be re installed.</li> </ul>
<b>Location</b>	<ul style="list-style-type: none"> <li>• Abutment of the dam</li> <li>• Any saddle along the reservoir rim</li> </ul>
<b>Discharge Capacity</b>	<ul style="list-style-type: none"> <li>• Usually &lt;1,000 m<sup>3</sup>/s; however, several references mention capacities up to 3,000 m<sup>3</sup>/s, but there are examples with more discharge (Bartlett Dam, USA, 7,425 m<sup>3</sup>/s)</li> <li>• Its participation usually represents a significant and fast increase of total discharge capacity of reservoir</li> </ul>
<b>Comments about design and functioning</b>	<ul style="list-style-type: none"> <li>• A fuse plug, even though is a supplementary work, must be carefully analyzed and designed.</li> <li>• Foundation: Erosion resistant material, better rock in medium to good physical condition. If considered, it can be treated or protected with specific solution as concrete slab, cut-off wall, concrete trench, rock's anchoring, etc.</li> <li>• Fuse plug's optimal height as per hydraulic and economic study; usually &lt; 8 m.</li> <li>• Dike section: zoned earth materials designed as a small earth dam, with a central or inclined clayey soil core, filters, shoulders and protections of granular soils or rock. Inclined core (Tinney-Hsu design, reference ?) is commonly used because process of failure is more reliable. Better a simple section, ease to build, and with predictable failure. Use of durable materials.</li> <li>• Provide a pilot channel on dyke to control wash out process. Core is the key element of the section for the washing out process.</li> <li>• Collapse of the fuse dike must progress gradually so its section should be designed for doing so. Time of failure should guarantee that reservoir level drops without endangering the dam. In cases, physical models are required. Numerical models can be used to assess the inundation areas</li> <li>• Crest level according to the design of service and/or auxiliary spillways. Overtopping occurs at extreme (low frequency) flood, usually ≥100 years return period flood</li> <li>• After failure, fuse plug is built again.</li> </ul>
<b>Comments about construction</b>	<ul style="list-style-type: none"> <li>• Site and watercourse should allow embankment's location and ability to pass the expected discharge on the downstream to the river course. Geology should guarantee a resistant (or limited) erosion material to avoid downstream channel and head cutting processes.</li> <li>• Length of dike less than 1,000 m is common.</li> <li>• Good site preparation for dike construction to avoid deleterious seepage.</li> <li>• Quality specifications for construction, with tolerances and to guarantee failure.</li> <li>• Materials for dike: clay, sand, gravel and small sized rock (rip-rap).</li> </ul>
<b>Highlights of this type of work</b>	<ul style="list-style-type: none"> <li>• Erosion: key process not only during wash out but also when water flows downstream through a natural stream or slope. Sometimes a protected canal is required.</li> <li>• Environmental impact should be considered: clogging of downstream channel, bank or slope erosion, existing facilities, potential damages to residents, and economical losses.</li> <li>• Surveillance and maintenance are required.</li> <li>• Access though dirty but stable road. Try to avoid remote location from dam.</li> <li>• Even though it could be seen as a secondary work, do not underestimate design criteria and constructions requirements.</li> <li>• Few fuse plugs have operated so very little factual experience is available</li> <li>• There are concerns about the "controlled failure" since earth dike can consolidate</li> </ul>

	<p>and becomes stronger, also vegetation can generate “soil armoring” with roots.</p> <ul style="list-style-type: none"> <li>• Fuse plugs have mainly be used in small and intermediate dams, but there are examples in large dams. They may be a very cost - effective solution.</li> <li>• According to USBR-FEMA (Wahl), topics that need research about fuse plugs are:             <ul style="list-style-type: none"> <li>✓ Long term performance of clay core which means long term reliability of plug</li> <li>✓ Cracking due to desiccation and settlement and possible deleterious leakage and piping</li> <li>✓ Use of other or new materials: Geo-membranes, Concrete core, etc.</li> </ul> </li> <li>• Extended use in several countries: China, USA, others.</li> <li>• Within the DRIP, an inventory and evaluation of existing dams with fuse plugs in India would be of particular interest for dam safety program.</li> <li>• For geotechnical criteria and recommendations see CWC Manual: “Assessing Structural Safety of Existing Dam”.</li> <li>• For aspects related to use of hydraulic models, see Annexure “D” of this Manual.</li> </ul>
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Table 3-9: Fuse Plug aspects



Figure 3-40: Keelit Dam, (New South Wales, Australia).



Figure 3-41: Timah Tasoh Dam, Malaysia

ERC reported the following description of the incident: “Late in the afternoon on Wednesday, May 14, 2003, high and turbid flows were observed in the Dead River several miles downstream of the remote Silver Lake Basin in Marquette County, Michigan. An operator was dispatched to the site and found that a fuse plug

*embankment, a feature of the project that is designed to fail sacrificially to prevent failure of more critical project works, had activated. The fuse plug embankment was entirely eroded away and erosion had progressed well into the discharge channel bottom and side slopes. The dam owner activated the emergency action plan and steps to protect downstream lives and property were initiated. During the subsequent 24-hour period, over 1700 residents were evacuated, several local road bridges and an abandoned railroad bridge were damaged or washed out, the City of Marquette’s Tourist Park dam near the mouth of the Dead River was overtopped and failed, the Presque Isle coal-fired power plant was shut down due to flooding, and two mines that rely on electric power from the power plant were shut down. There was extensive erosion of the river banks and significant impacts to the Dead River fishery. No loss of life or personal injuries occurred”.*

Later technical evaluation highlighted these findings: (1) Incident responded to a 5 to 10 years storm, (2) Pilot channel on fuse plug had its invert lower than service spillway (9 inches), rendering service spillway unable to function, (3) Local foundation material and along downstream channel was highly erodible resulting in head cutting which progressed toward reservoir, during high frequency flood, and (4) Down cut almost emptied the reservoir (ICOLD, J.P. Tournier et al, 2019). The main lessons learned from this incident were: (1) Inadequate fuse plug design, poor geology at site, (2) A owner’s

Emergency Plan available for local government and residents, and its coordination, was vital for controlling consequences and (3) Accurate inundation maps were invaluable tools (T. Schwalbach, Emergency Management Coordinator for Marquette County, Michigan) See Figure 3-48.

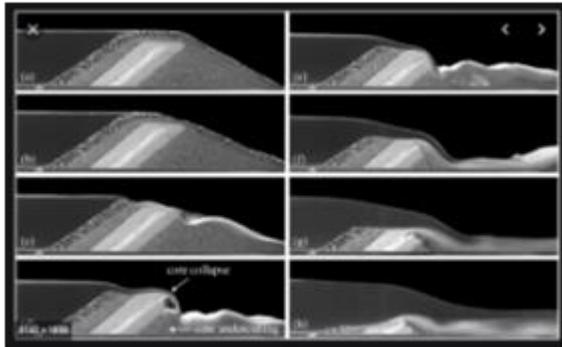


Figure 3-42: Typical mode of failure of a fuse plug during operation.



Figure 3-43: Strontia Dam (Colorado, USA).

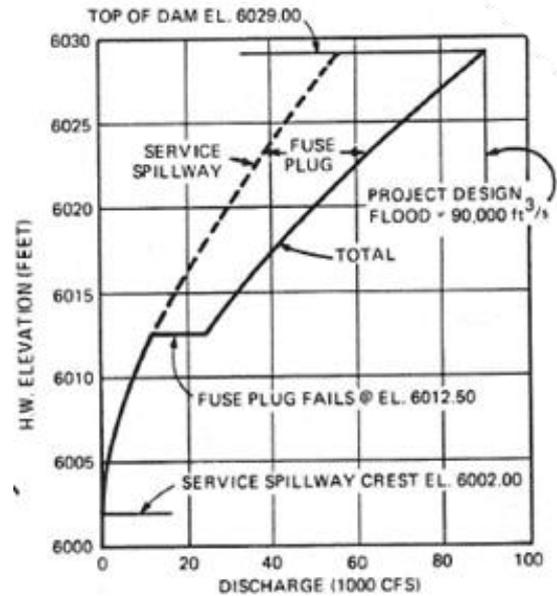


Figure 3-45: Strontia Springs Dam (South Plate river, Colorado, USA, (Coleman and Wei)



Figure 3-46: Bartlett Dam, (USA).



Figure 3-44: Strontia Springs Dam (South Plate river, Colorado, USA), fuse plug on left abutment. (Coleman and Wei).



Figure 3-47: Fuse plug rebuilt. Silver Lake Dam, Dead River, Michigan, USA (FERC).

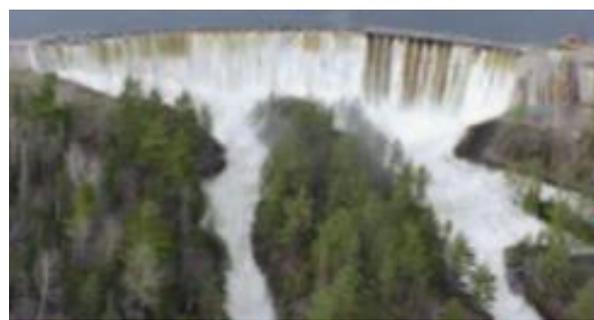


Figure 3-48: Downstream consequences of fuse plug functioning and local erosion failure at reservoir rim. Consequences: River banks erosion and overflow of concrete dam downstream without damages (T. Schawalbach)

## 2. Labyrinth weir

While developing options for increasing discharge capacity, the increase in length has been the classic approach because of its greater influence than other parameters. There are several types of improved weirs with curved and wavy axis, such as fan, bath tub and others; however, for larger discharges, research has been focused to irregular or zigzag axis. Labyrinth weir has a geometrical layout in folded units or cycles that allows a significant increase in length in the available width/space.

As defined by Khatsuria (2005): “The labyrinth weir consists of a series of relatively slender walls having a repetitive plan form, shaped generally triangular or trapezoidal, with a vertical upstream face”. By using this weir, the increase in length has an important effect in discharging capacity even though coefficient of discharge is lower (due to hydraulic performance) than lineal weir. The weir is integrated by several overflow walls (drop structures) of uniform height placed on a flat bottom. Plan layout is symmetrical with different shapes: triangular, rectangular or trapezoidal. The trapezoidal shape is the latest and the most used option.

This type of control structure can be used for new spillways as well as for rehabilitation of existing spillways for improving the hydraulic safety of the dam when discharge capacity is insufficient. The performance of labyrinth weir depends on hydraulic conditions and geometry. For each combination of characteristics there will be an optimal solution. In case of rehabilitation

of an existing dam, labyrinth spillway is to be planned in the available space only. Figure 3-49 shows a scheme of a labyrinth weir with possible crest shapes of walls and overflow condition.

A labyrinth behave as a slender weir with shaped crest (usually rounded) with free overflow or little nappe interference at apexes according to relation between energy head upstream ( $H_T$ ) and height of the wall ( $P$ ).

The discharge’s equation has the following expression and notations (for SI units):

$$Q = \frac{2}{3} * C_{d(\alpha)} * L_c * \sqrt{2g} * H_T^{3/2}$$

$Q$  = Discharge ( $m^3/s$ )

$L_c$  = Centerline length (m) along the whole weir:  $L_c = N(2l_c + A + D)$

$A$  = Inside apex length

$D$  = Outside apex length

$C_{d(\alpha)}$  = Dimensionless discharge coefficient ( $m^{1/2}/s$ ). See Figures 3-49, 3-50 and 3-51.

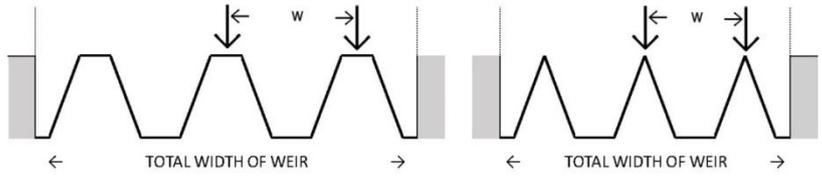
$H_T$  = Total upstream hydraulic head (unsubmerged) measured relative to crest level ( $H_T = h + v^2/2g$ , where “ $v$ ” is the average cross-sectional velocity at the location and “ $h$ ” is the piezometric head upstream of the weir relative to the weir crest elevation) (m)

$N$  = Number of labyrinth cycles

For labyrinth’s hydraulic design, physical modeling is recommended. (see Appendix “D”).

Table 3-10 presents a summary of aspects related to use, design, construction and control of labyrinth weir. Figures 3-52 to 3-

60 shows examples, typical sections and details of this weir as defined.

<p><b>Plan view and arrangement of labyrinth weir</b></p>	 <p>Concrete weir trapezoidal or triangular pattern with rectilinear or curved axis, limited height.</p>
<p><b>Use</b></p>	<ul style="list-style-type: none"> <li>• Service spillway</li> <li>• Emergency spillway</li> <li>• Typical use for rehabilitation of existing spillways with inadequate capacity.</li> <li>• For any type of dam: Concrete, Rock fill, Earth fill or Combination.</li> </ul>
<p><b>Location</b></p>	<ul style="list-style-type: none"> <li>• Abutment of the dam</li> <li>• On the dam, for gravity dams</li> <li>• Over existing spillway to be rehabilitated</li> </ul>
<p><b>Discharge Capacity</b></p>	<ul style="list-style-type: none"> <li>• Usually &lt; 3,000 m<sup>3</sup>/s or specific discharge less than 50 m<sup>3</sup>/s/m; however, it has been used for up to 25,000 m<sup>3</sup>/s</li> <li>• Specific discharge (q) usually doubles that of lineal rounded crest weir “W”, for same water head, but this increase varies with geometrical design.</li> <li>• Design Flood: Selected IDF or greater flood according to Dam Safety.</li> <li>• Can increase active storage of reservoir by using part of freeboard without affecting security</li> <li>• Capacity could drop due to obstruction by floating debris, but there are no reported cases.</li> </ul>
<p><b>Comments about design and functioning</b></p>	<ul style="list-style-type: none"> <li>• Foundation: Rock or concrete body of existing spillway or dam</li> <li>• Optimal height as per hydraulic, economic study and constructability (max. reported 9 m).</li> <li>• Each cycle has vertical wall with rounded (or designed shape) crest; common shapes are half and quarter round; shape could be more efficient but design also looks for ease in construction.</li> <li>• Research on physical models gives weir ratios and parameters for best hydraulic performance. Typical installation for trapezoidal layout uses these ratios(see Figure 3-51 for notations):             <ul style="list-style-type: none"> <li>- <math>H_T/P \leq 0.9</math> varies with <math>\alpha</math> and crest shapes (range of models <math>0.05 \leq H_T/P \leq 0.9</math>), common initial design value <math>H_T/P = 0.5</math></li> <li>- <math>w/P = 2</math> to 4, currently this ratio has been superseded by other called Interference Ratio which relates Length of Effective Disturbance close to apex with length of weir (LED/L) that modifies the coefficient of discharge (width of one weir cycle = w).</li> <li>- <math>M = L_c/N.w &lt; 9.5</math> (<math>L_c</math> = Centerline length along the whole weir , <math>N</math> = number of weir cycles), this geometrical ratio is called Magnification ratio; currently it is used to define another ratio called Efficacy (e) that relates coefficient of discharge of labyrinth weir (<math>C_{dq}</math>) with that for lineal weir (<math>C_{d90}</math>, with length W) so: <math>e = [C_{dx}/C_{d90}]M</math>, that indicates the combined effects of less coefficient of discharge and larger length, of labyrinth weir.</li> <li>- <math>A = t_w</math> (trapezoidal)</li> <li>- <math>A/w &lt; 0.08</math></li> <li>- <math>\alpha = 6</math> to 35 degrees</li> </ul> </li> </ul>

	<ul style="list-style-type: none"> <li>• Other important aspects for improving efficiency are: length of zones of disturbances close to wall apexes and submergence due to downstream water level.</li> <li>• Increasing head over the weir reduces efficiency (lower discharge coefficient and greater disturbances of flow)</li> <li>• Latest standard design practice (Crookston and Tullis, 2013).</li> <li>• Physical model is recommended.</li> <li>• Crest level at Normal Operation Pool Level. Could be used to increase reservoir volume</li> </ul>
<b>Comments about construction</b>	<ul style="list-style-type: none"> <li>• Volume of concrete is less than surface overflow ogee weir</li> <li>• Quality specifications for construction. Forming and finishing of concrete surfaces add cost.</li> <li>• For rehabilitation of existing spillways, required area of implantation (Area = WB) must be well defined at design phase, and later when estimating cost of construction. In some cases the required space could result in a drawback for using this weir.</li> </ul>
<b>Highlights of this type of work</b>	<ul style="list-style-type: none"> <li>• Investigation is still ongoing for optimizing weir design and for cost efficient solutions. Aspects under research are: aeration, nappe instability, higher heads, crest shape, flow approach condition, layout and floating debris.</li> <li>• Accumulation of floating wood debris could reduce spillway capacity.</li> <li>• For spillway rehabilitation, capacity of downstream chute must be verified and also its freeboard.</li> <li>• In some cases this structure cannot be placed on concrete gravity dam due to required dimensions of footprint area</li> <li>• No reported incidents or malfunctioning of labyrinth weirs.</li> <li>• Discharge is high since the beginning of flood, so downstream channel capacity and flood prone zones must be evaluated as a part of an emergency plan.</li> </ul>

Table 3-10: Labyrinth Spillway aspects

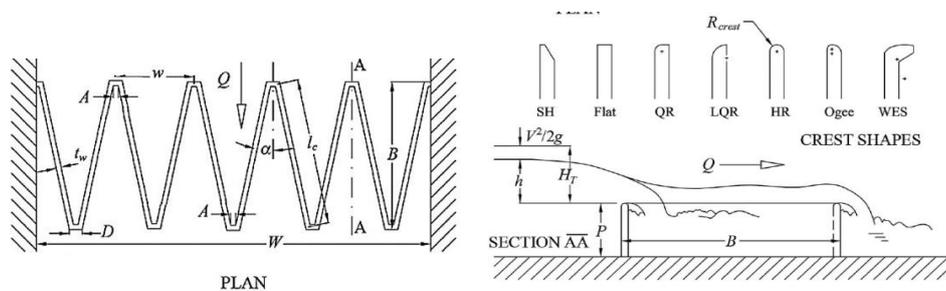


Figure 3-49: Labyrinth weir: Geometric parameters and flow pattern (Crookston and Tullis, 2013 in ICOLD 2016).

Figures 3-57 to 3-58 show a labyrinth spillway over a concrete dam (Midmar dam)

**Examples of Labyrinth weir**

a) Labyrinth considered in original dam’s design

Town head dam (new), (Water Supply Greensboro, North Carolina, USA) (see Figure 3-52, 3-54 to 3-56). Embankment

dam, 15 m high, built in 2012. Reservoir capacity: 28.3 hm<sup>3</sup>. This dam, even though is a new project, corresponds to the rehabilitation of an existing dam located closely upstream (built in 1969). In the new construction the Service labyrinth spillway increased the discharge capacity to 2,320 m<sup>3</sup>/s; according to dam safety standards for IDF (4,050 m<sup>3</sup>/s) an emergency spillway was also included over the dam (slope protection). The old gravity dam spillway did

not guarantee the structural safety due to severe damage from alkali-silica reaction in the concrete, so it needed to be replaced; the dam classifies as High Hazard Large Dam. This project was awarded for engineering quality and excellent concept of rehabilitation (2012 and 2013, Schnabell Engineering).

b) Labyrinth was included during rehabilitation

Ute dam; (NMISC, USBR); Canadian river, New Mexico, USA. Embankment dam, 45 m high, built in 1963. Service labyrinth spillway

was added to rehabilitate and upgrade existing lineal free overflow spillway and to increase reservoir storage (1984).

The labyrinth spillway has 14 cycles with walls 9.14 m high and operates for IDF discharge of 16,042 m<sup>3</sup>/s. It is one of the largest labyrinth spillways in USA. Figure 3-59 shows a view of finished spillway and Figure 3-60 its discharge curve for the labyrinth of 14 cycle, triangular shape, with H/P=0.74 and maximum water head of 6.75 m .

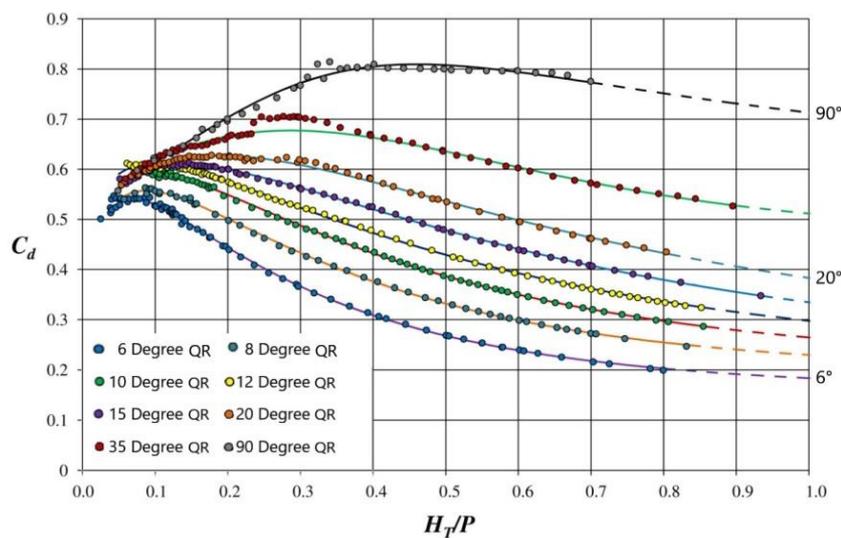


Figure 3-50: Values of  $C_{d(\alpha^\circ)}$  versus  $H_T/P$  for quarter-round trapezoidal labyrinth weirs

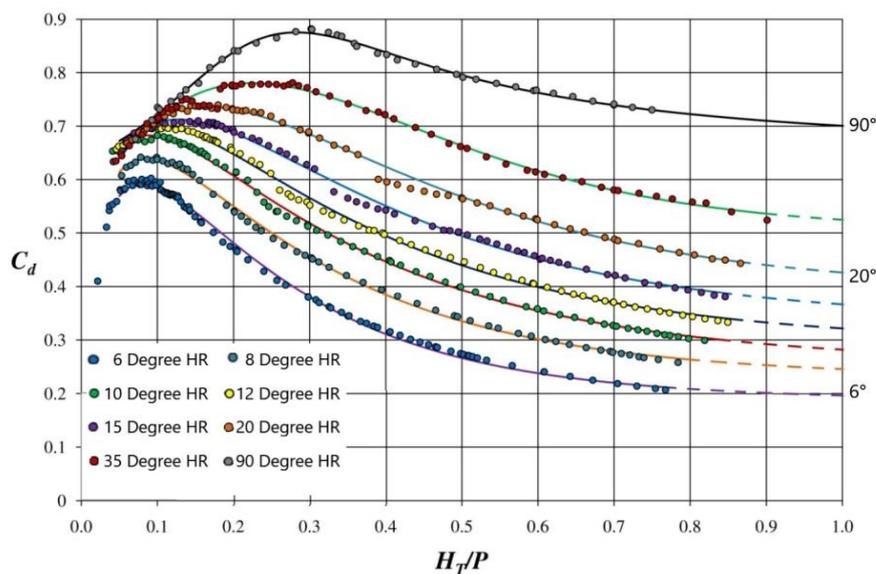


Figure 3-51: Values of  $C_{d(\alpha^\circ)}$  versus  $H_T/P$  for half-round trapezoidal labyrinth weirs



Figure 3-52: Townhead dam. Labyrinth weir with 7 cycles (Schnabel Engineering)



Figure 3-53: Midmar dam (Soth Africa)



Figure 3-54: Townhead dam. Labyrinth weir. Phases of construction and demolition of old spillway(Schnabel Engineering)



Figure 3-55: Townhead dam. Labyrinth weir. New dam located closely downstream of old dam(Schnabel Engineering)

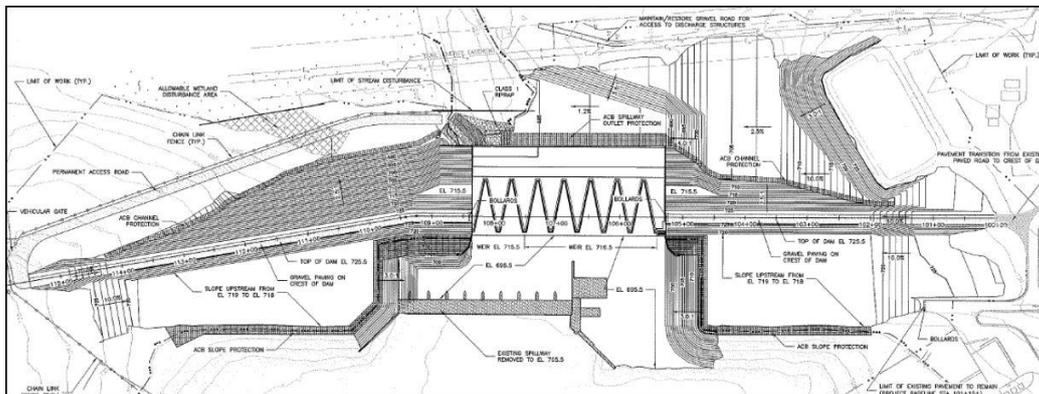


Figure 3-56: Townhead Dam, (North Carolina, USA). Plan view (Schnabel Engineering).

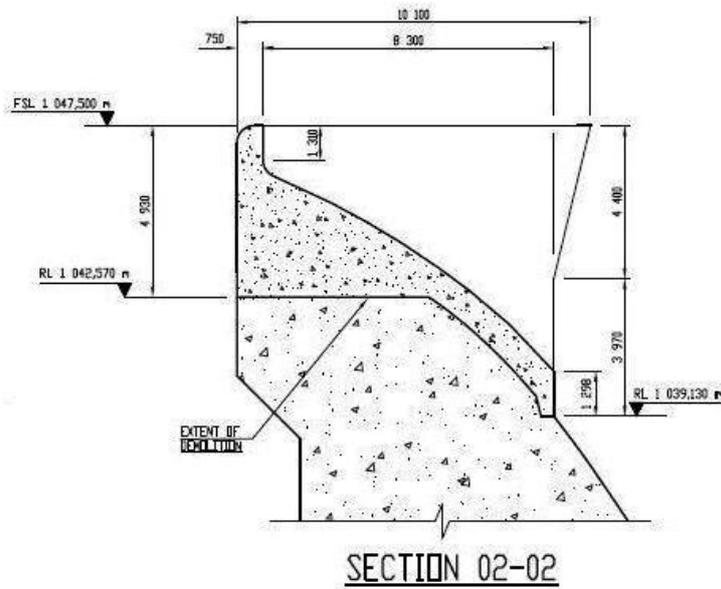
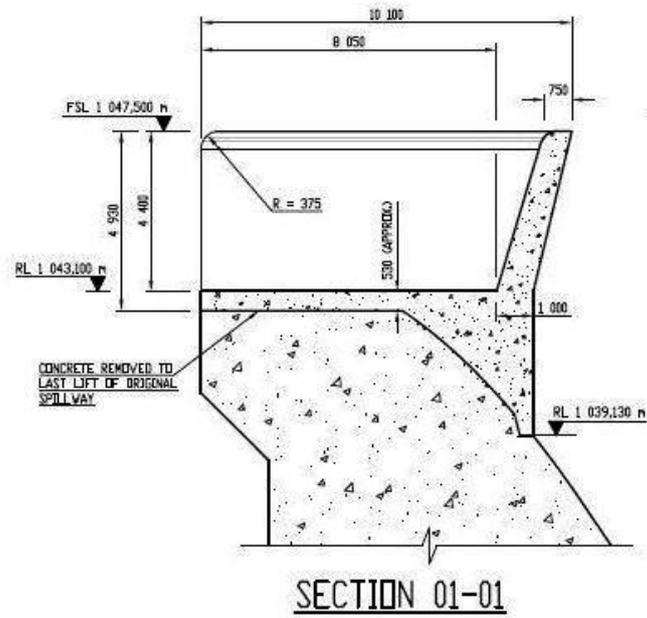


Figure 3-57: Midmar Dam Labyrinth Spillway Sections (South Africa)

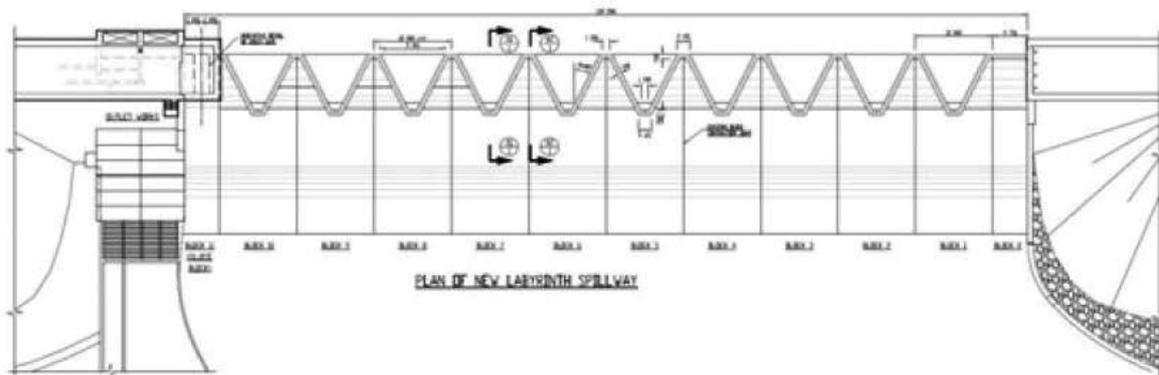


Figure 3-58: Midmar Dam Labyrinth Spillway Layout (South Africa).



Figure 3-59: Ute Dam, USA (USBR, 1982)

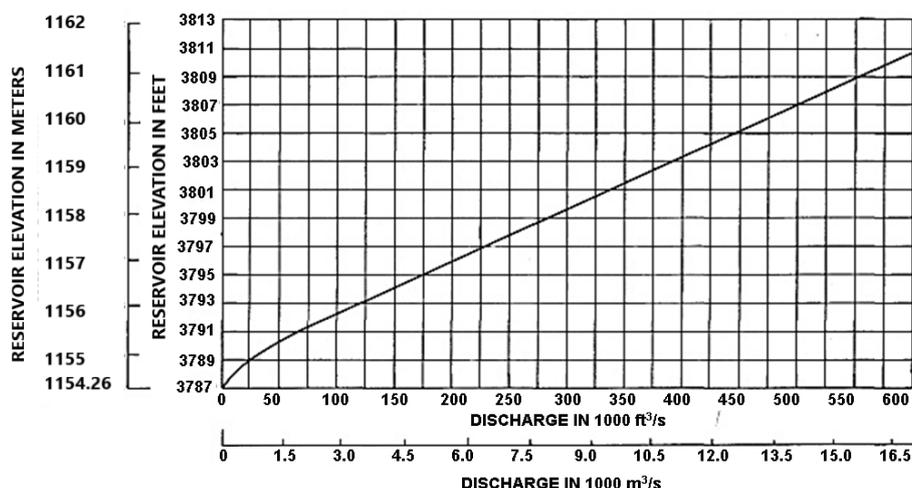


Figure 3-60: Ute Dam, Labyrinth spillway's Discharge Curve (USBR, 1982)

### 3. Piano Keys Weir (PKW)

ICOLD defines PK weir as a new option for increasing discharge capacity and/or storage capacity. It is considered as a variation of traditional labyrinth weirs. PKW was first proposed by Hydro Coop in collaboration with the Hydraulic Laboratory of Electricité (France), Roorkee University (India) and Biskra University (Algeria) (Ouamane and Lempérière). It has been implemented in many dams in the last 15 years. The first PKW is operating since 2006 in France.

Some of its advantages are:

- Small footprint: Required space for its location is reduced because structure

incorporates overhangs. Compared to a rectangular labyrinth layout, PKW is easier to install, specially, at sites having limited foundation space (e.g., crest of a gravity dam).

- Efficiency: Greater than traditional labyrinth weirs since total length of weir is larger because of overhangs. Higher unit discharges for low heads; for which it works as a free overflow in almost whole length.
- Reliability: As an overflow free weir it is more reliable than controlled weirs with gates.

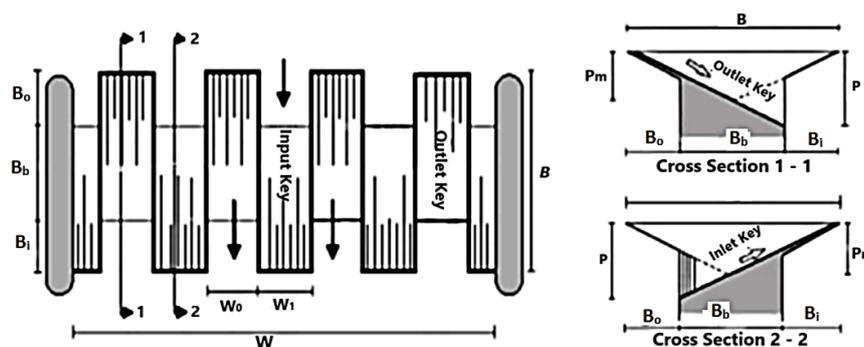


Figure 3-61: PKW typical layout and section (University of Biskra).

This type of control structure can be used for new spillways (service or auxiliary) and for rehabilitation of existing spillway for improving hydraulic/hydrologic dam safety when discharge capacity is insufficient. At their peak efficiency the PK weirs can allow specific discharges of up to  $100 \text{ m}^3/\text{s}/\text{m}$  (Ouamane and Lempérière), although in practice the maximum discharge values are usually of the order of  $20 \text{ m}^3/\text{s}/\text{m}$  for a depth of 2 m (Laugier et. al, .2017). This is typically between 2-4 times higher than that for a linear weir at a similar hydraulic head. These characteristics allows its use in rehabilitation of gravity dams. There are examples where a combined solution has been adopted, PKW as service spillway and controlled (radial gates) as auxiliary spillway.

It has a rectangular layout with ramped floors which create overhanging or cantilevered apices and with smaller footprint area than the labyrinth weir. The ramped floors reduce the vertical walls height and thus the volume of reinforcing steel required in concrete, which is an improvement of typical uniform wall's height in a labyrinth weir. The succession of inclined apices alternatively, as outlets and inlets (keys), with overhang in upstream and in downstream directions, gives the name Piano Key weir. Figures 3-61 shows a scheme of a PKW and a unit with common notations as:

- Sub-indices: i = inlet and o = outlet
- B = length of side walls (stream wise, m)
- W = total (straight) width of weir (m)
- $w_i$  = Width of inlet (m)

- $w_o$  = Width of outlet (m)
- P = Height of weir (m)
- L = Length of unit =  $2B + w_o + w_i$  (m)
- t = Wall thickness (m)
- $w_u$  = Width of one unit =  $w_o + w_i + 2t$
- $N_u$  = No. of units
- L = Total crest length of PKW weir  
=  $N_u \times L_u$

Flow over a PK weir is complex; it depends on hydraulic head, flow approach pattern and geometric parameters; for each combination of characteristics there will be an optimal solution; for rehabilitation of an existing work, even though available space is limited, PKW's length can be adapted to meet the desired purpose.

For low head, PKW performs almost as a free over flow weir and unit discharge is large, but, as head increases, flow becomes 3-D, so flow pattern is no longer perpendicular to weir crest, and efficiency decreases. In order to reach an optimal solution, physical modeling is required since no standard design procedure is available. Extensive research on PKW has been carried out, in several countries: France, Algeria, India, Australia, Vietnam, Switzerland etc.

Equation for discharge over a PKW, for low head, can be defined according to two approaches based on length of weir or unit length (L) or width of weir (W). The expression proposed by Ouamane & Lempérière (2006) is:

$$Q_p = C_{p,W} * W * \sqrt{2 * g * H^3}$$

$Q_p$  = Discharge (m<sup>3</sup>/s)

W = Width of weir (m)

H = Upstream hydraulic head (m)

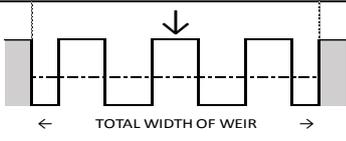
$C_{p,W}$  = discharge coefficient of PK weir as related to W. It takes into account effects of efficiency.

The discharge coefficient,  $C_{p,W}$  can be estimated from the physically measured, or numerically modeled discharge (Pfister & Schleiss 2013b). The W subscript of the discharge coefficient refers to the notion that the discharge being estimated uses the linear width of the PK weir and not that of the overflow crest length as a whole, which can be many multiples higher than the width. The benefit of the longer crest is thus reflected in the discharge coefficient.

Discharge per unit width W of a PKW compared to an straight ogee crested weir is presented in Figure 3-64.

PK weirs have been developed and modeled for four types of units, Figure 3-62 and Figure 3-63 show arrays and names assigned; the basic difference is the inclusion or not of overhangs in one or both directions or none. Type A has symmetric overhangs both u/s & d/s, Type B has single u/s overhang, Type C has single d/s overhang & Type D is without overhang. Types A and B are commonly used.

Table 3-11 presents a summary of aspects related to use, design, construction and control of PKW. Figures 3-65 to 3-72 show examples, model, typical sections and details of PK Weir.

<b>Plan view and arrangement for PK weirs</b>	Concrete weir, labyrinth in a rectangular layout with rectilinear or curved axis, limited height.	
<b>Use</b>	<ul style="list-style-type: none"> <li>• Service spillway</li> <li>• Auxiliary spillway</li> <li>• Typical use for rehabilitation of existing spillways with inadequate capacity.</li> <li>• For any type of dam: Concrete, Rock fill, Earth fill or Combination.</li> </ul>	
<b>Location</b>	<ul style="list-style-type: none"> <li>• Abutment of the dam</li> <li>• On the dam, for Gravity or Arch dams</li> <li>• Over existing spillway to be rehabilitated</li> </ul>	
<b>Discharge Capacity</b>	<ul style="list-style-type: none"> <li>• Usually 100 to 2,000 m<sup>3</sup>/s; however, they have been for capacities up to 10,000 m<sup>3</sup>/s</li> <li>• Unit discharge up to 20 m<sup>3</sup>/s/m: however, recent designs extend this upper limit to 70 m<sup>3</sup>/s/m</li> <li>• This advance of labyrinth weir increases capacity in relation to former layouts</li> <li>• Design Flood: IDF or greater according to Dam Safety</li> </ul>	
<b>Comments about design and functioning</b>	<ul style="list-style-type: none"> <li>• Foundation: Same as for Ogee and Chute spillway or Concrete body of existing spillway or dam. i.e. Good rocky foundation.</li> <li>• PKW optimal height as per hydraulic and economic study</li> <li>• PKW design could be different for new spillways and for rehabilitation works, because limitation of space affects layout and geometry of weir, also limitation in weir height (P).</li> <li>• Each unit has vertical walls and rounded (or designed shape) crest</li> <li>• Units of weir are four types: A, B, C and D. Most common types are</li> <li>• A and B</li> <li>• For rehabilitation of spillways with a PKW Type “A” some typical ratios are:                         <ul style="list-style-type: none"> <li>– In relation to equivalent lineal Creager weir “W”, discharge capacity usually increases 2 to 5 times (common 3 fold) for low head (H/P &lt; 0.5) and 1.5 times for high heads (H/P &gt; 1)</li> </ul> </li> </ul>	

	<ul style="list-style-type: none"> <li>- Length of weir (walls) is almost 4 to 7 times the length of linear spillway (L/W)</li> <li>- A high PKW (<math>P/W_u = 1.30</math>) is adopted in new projects being hydraulically more efficient. For dam rehabilitation <math>P/W_u = 0.50</math> (approx.) is more practical, though less efficient.</li> <li>- Unit discharge (<math>q</math>) close to <math>4.3 H \sqrt{P}</math> (<math>m^3/s/m</math>); <math>H</math> = Head on weir and <math>P</math> = weir's height</li> <li>- Head over a PKW is around <math>\frac{1}{2}</math> the head of lineal Creager weir for same discharge</li> <li>- Width of inlet and outlet overhangs (<math>w_i</math> and <math>w_o</math>) could be equal or inlet wider by 20%</li> <li>• There are no standard design practice. Physical modeling is required.</li> <li>• Crest level at Normal Operation Pool Level. Could be used to increase reservoir volume</li> </ul>
<b>Comments about construction</b>	<ul style="list-style-type: none"> <li>• Quality specifications for construction. Forming and finishing of concrete surfaces add cost.</li> <li>• Several advantages of the structure: (a) Lesser volume of reinforced concrete than labyrinth, (b) Smaller footprint area so it is easier to install for new and rehabilitated spillways.</li> </ul>
<b>Highlights of this type of work</b>	<ul style="list-style-type: none"> <li>• Attractive solution for new spillway with capacity in its range of operation but research is ongoing for greater unit discharges.</li> <li>• Highly reliable, no mechanical elements, no need of operation.</li> <li>• Investigation is still ongoing for optimizing geometry and design. Aspects under research are: Aeration, Energy dissipation, Flow approach condition and Floating debris.</li> <li>• Application to rehabilitate ogee crest and chute spillways.</li> <li>• Intense research (last 10 years)</li> <li>• It has been used as service spillway in combination with a controlled auxiliary spillway</li> <li>• There are special design where PKW has been adapted to shaft (Morning Glory) weir.</li> <li>• Discharge is high since the beginning of flood, so downstream channel capacity and flood prone zones must be evaluated as a part of an emergency plan.</li> </ul>

Table 3-11: Piano Keys Weir (PKW) aspects

### Examples of PKW

#### a) PKW considered in original dam's design

Van Phong Dam, Vietnam. Concrete dam, 18 m high, built in 2012. Combination of spillways: a section controlled with 10 radial gates plus 2 sections of PKW, total discharge 15,350  $m^3/s$  out of which the PKW discharge was 8,700  $m^3/s$  (see Figure 3-69).

Another example is Xuen Minh Dam, Vietnam. With a Type B PKW with following features: 21 inlets and 21 outlets, total spill's

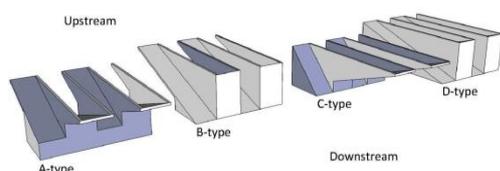


Figure 3-62: Types of PKW units (Epicum et al).

length,  $L = 894$  m [ $L/W=6$ ]. The array is a combination of PKW as service spillway and two radial gates as auxiliary spillway. Total discharge 11,900  $m^3/s$ , PKW discharge: 9,700  $m^3/s$  (see Figure 3-68).

#### b) PKW included during rehabilitation of dam

Saint Marc Dam, France. Concrete gravity dam, 40 m high, built in 1936, Reservoir capacity, 20  $hm^3$ . For dam security the

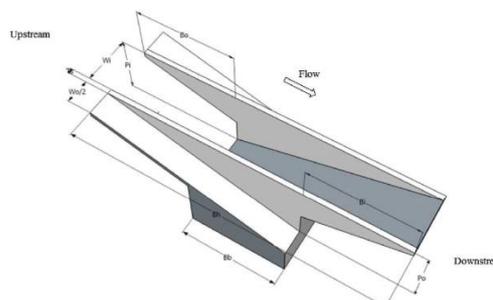


Figure 3-63: PKW typical unit (ICOLD).

auxiliary spillway, controlled by two radial gates, was supplemented with a PKW (2008) for a discharge of  $135 \text{ m}^3/\text{s}$ .

For installation, the crest of dam on right side of controlled spillway was demolished

to allow the required space (15.6 m) for a new PKW with a  $L/W = 5$ ,  $P = 5 \text{ m}$ ,  $B = 12 \text{ m}$ , walls with half rounded crest. Construction was completed in 5 months (Laugier et al.) (see Figures 3-71 and 3-72).

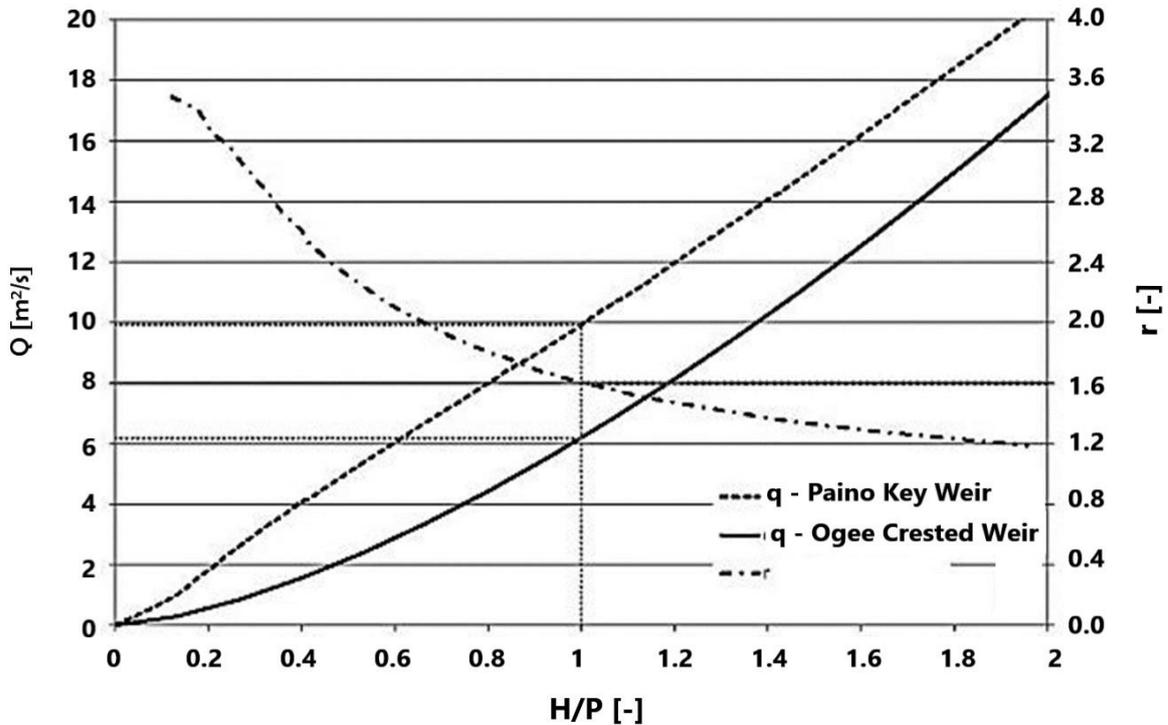


Figure 3-64: Total discharge coefficients for various weir types (Blancher et al. 2011)

(Note: Discharge increase ratio =  $r = C_{p,w} / C_s$ .

$C_s$  = Discharge coefficient for standard linear weir)



Figure 3-65: PKW types with overhangs and without them.



Figure 3-66: Typical physical model PKW's functioning (Epicum et al).

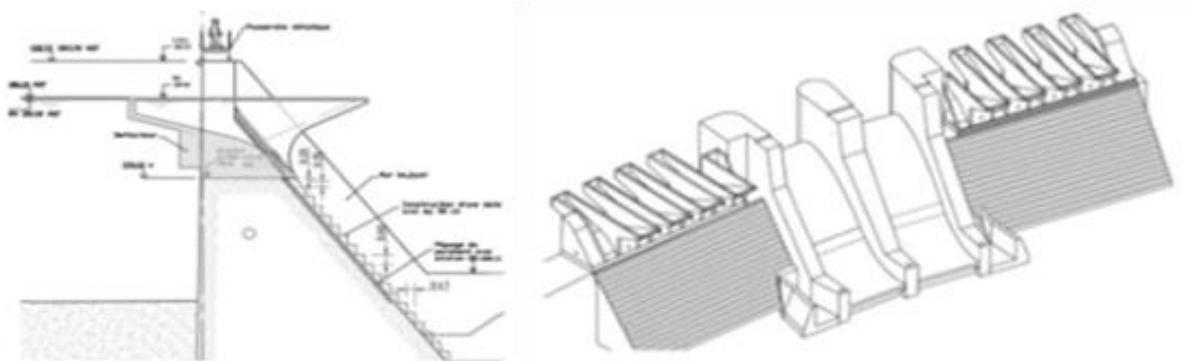


Figure 3-67: PKW combined with free ogee crested to enlarge an existing spillway at Charmine Dam, France.

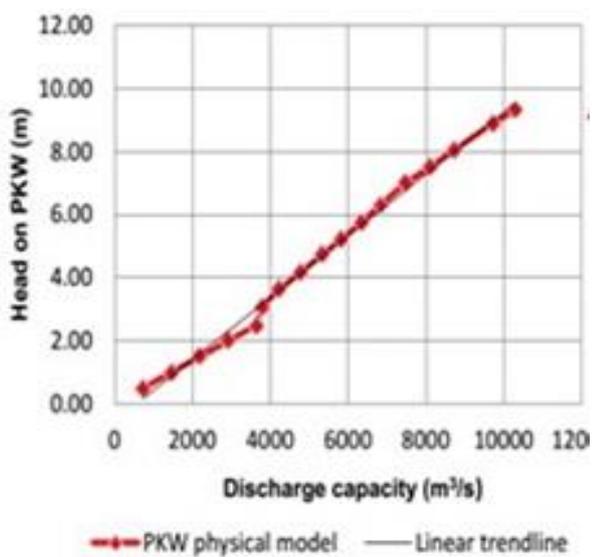


Figure 3-68: Discharge's curve of a PKW (Type B) Xuan Minh Dam, Vietnam (2016).



Figure 3-69: Van Phnog Dam, Vietnam.

PKW discharge: 8,700 m<sup>3</sup>/s. In Vietnam there are several dams with PKW's with ranges of discharges from 1,000 to 9,700 m<sup>3</sup>/s.



Figure 3-70: PKW on a Morning Glory weir to increase spillway capacity. Our Black Esk Dam (USA).



Figure 3-71: Saint Marc Dam, France. ( after and before, rehabilitation)

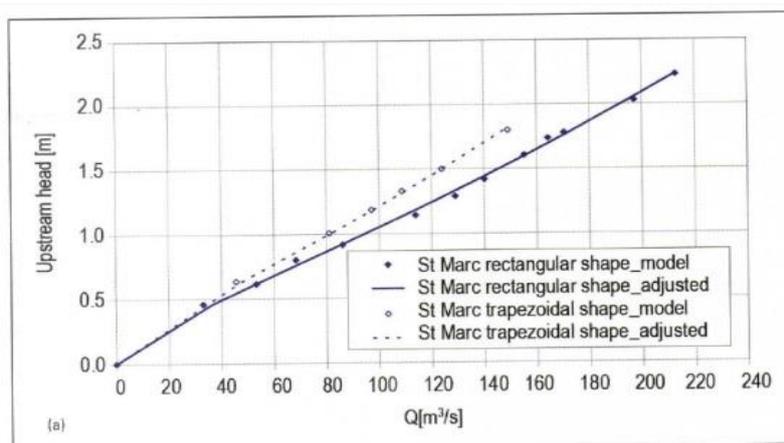


Figure 3-72: Rating curve as in physical model – Saint Marc Dam, France.

#### 4. Fuse gates

Fuse gate utilizes a fuse plug mechanism for its working and can be used as a service or auxiliary spillway. A typical installation consists of multiple gates placed over a spillway crest (brought well below FRL in case of existing dam), with a shape of a labyrinth weir in which each gate represents one cycle of the labyrinth (Falvey and Treille). According to its assigned function, it can be used to either increase the spillway discharge capacity and/or to improve storage capacity, by fixing its crest at normal water level i.e. at FRL or higher. Fuse gates is a trademark patented by Hydroplus International (France; 1991). They can be used both in new dams and for rehabilitation of existing dams, as alternative to radial gates or fuse plug. Figure 3-73 shows the design of Fuse Gate and its standard components.

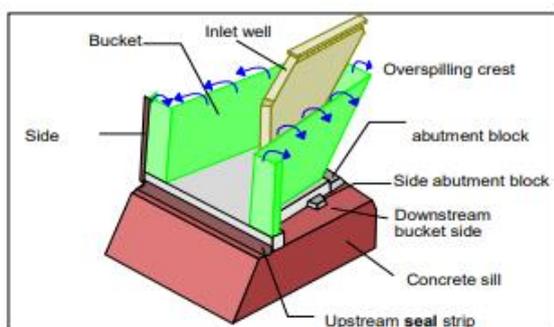


Figure 3-73: Typical Fusegate (Patel et al., 2009)

Fuse gates consist of units fabricated of concrete or steel; these units are placed side by side at site and they withstand hydraulic forces by gravity. Their operation depends on reservoir levels.

For low to moderate head over its crest it behaves as a labyrinth weir. For discharges greater than the design flow, water begins to flow through the well through an opening in it and into the chamber located at the base of the gate in which drain holes are provided. The level of this opening can be at different designed levels in different fuse gate units. When the inflow into the well exceeds the flow out of the drain holes below, the water level in the well increases. This causes the pressure in the bottom chamber to increase and an uplift force is exerted on the gate decreasing its stability. For a particular depth of water in the well or reservoir level the gate becomes unstable and tilts by rotating about its downstream edge. After a predetermined reservoir level each gate unit overturns in progressive fashion, by rotating about its downstream edge; so downstream space required for tilting of gates has to be adequate; finally, the units fall down and have to be removed. It is important to mention that fuse gate operation can be controlled to within a few centimeters of head. Figure 3-74 indicates the functioning and tilting process.

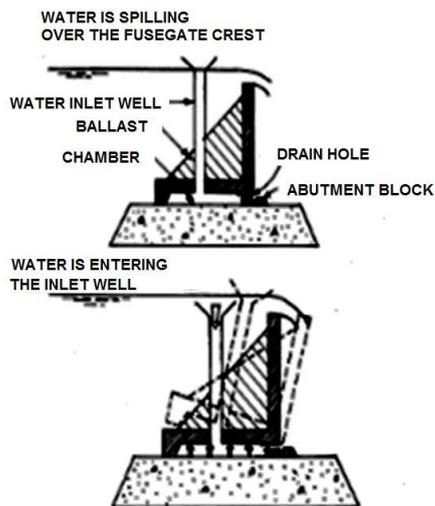


Figure 3-74: Functioning of a unit (Ait Alla).

Units are fabricated and are commercially available of standard shape and dimensions, and for different tilting heads. There are several models with predefined heights. For classical fuse gates, once they overturn, usually they cannot be re-used because they get damaged after falling. Also, they can damage the spillway glacis/chute. However, their design and functioning have since been improved so there are models which allow for their reuse. Figure 3-78 shows components, the schemes and the dimensions of the three standard design of fuse gate units and installation on existing spillway.

In India (Gujarat state), there are, at least, 15 spillways with fuse gates. Wanakbori Weir (first in India; 1995) with 33 concrete fuse gates (with a maximum registered discharge of 32,590 m<sup>3</sup>/s). In other three of these reservoirs: Dhatarwadi (earth fill dam, 25 m high, spillway capacity of 4,342 m<sup>3</sup>/s), Sonmati (concrete dam, 17 m high, spillway capacity of 1,039 m<sup>3</sup>/s) and Sorthi (no data available); fuse gates operated during flash floods with significant water volume (Solanki, Shirimali and Gandhi).

Flow over fuse gates corresponds to a weir which functions according to water elevation and state of the unit: (a) Before tilting, with water elevation above fuse gate crest but less than tilting head, as a labyrinth weir; (b) After tilting of the unit, the flow

corresponds to that on a flat horizontal sill, acting as a sharp, short, broad or long weir according to the head and length of weir. So its hydraulic performance depends on hydraulic head and geometric parameters, which has been broadly studied in research on physical models; thus fuse gate has become a viable option for spillways.

Discharge over the weir for condition “Before tilting” has the expression and notation (in SI units) as under:

$$Q = \frac{2}{3} * C_d * L_f * \sqrt{2g} * h^{3/2}$$

Q = Discharge (m<sup>3</sup>/s)

L<sub>f</sub> = Crest length of fuse gate weir (m)

C<sub>d</sub> = Discharge coefficient (non-dimensional). This may be calculated from technical references: Falvey and Treille (1995), (Khatsuria, 2005) or according to manufacturer. Figure 3-75 shows C<sub>d</sub> for standard configurations of fuse gates.

h = head of overflow (m) = Elevation of reservoir – Elevation of crest of the fuse gate

H = height of fuse gate (m)

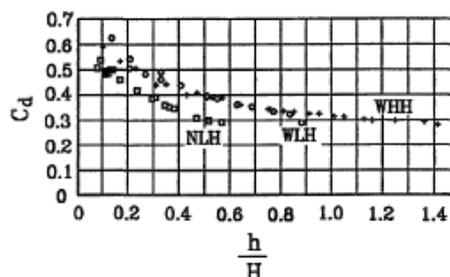


Figure 3-75: Discharge coefficients for standard Fuse Gates (Falvey and Treille).

Discharge for condition “After tilting”, with flow through space between fuse gates will correspond to flow over broad crested weir and is given by the following expression.

$$Q = 1.705 L_s H_0^{3/2}$$

where,

L<sub>s</sub> = width of the flow passage

H<sub>0</sub> = head over the spillway crest = Upstream energy line minus sill elevation (See Figure 3-76).

Some researchers indicate that in this case, the space between gates behaves as a “broad crested weir with short length,” with weir crest ratio of  $0.4 \leq H_0/W_c \leq 1.0$  (critical depth at crest and  $C_d = 1.705$  in SI units). (See Figure 3-76). For other weir crest ratios the coefficient of discharge may be evaluated as given in above mentioned references; however, hydraulic models are recommended to evaluate discharge performance.

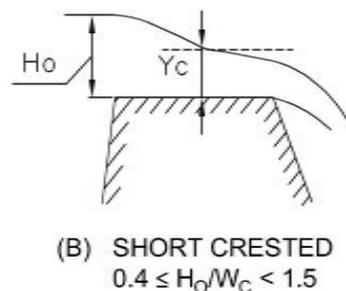


Figure 3-76: Broad crested weir (Khatsuria,2005)

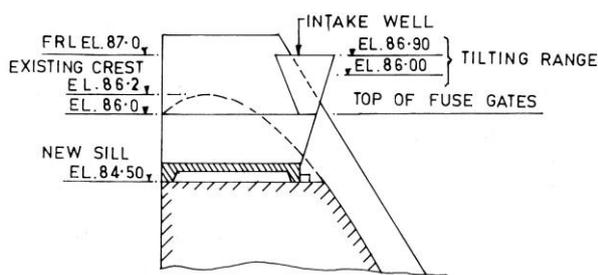


Figure 3-77: Installation of fuse gate on an existing spillway (flat/horizontal surface) (Khatsuria, 2005)

Figures 3-77 is an scheme of installation of a Fuse Gates on existing spillways. Figures 3-79 and 3-83 show examples of spillways with this installation. Table 3-12 presents a summary of aspects related to use, design, construction and control of fuse gates for new and rehabilitation of existing spillways.

**Examples of Fuse gates weir**

a) Fuse gates considered in dam’s

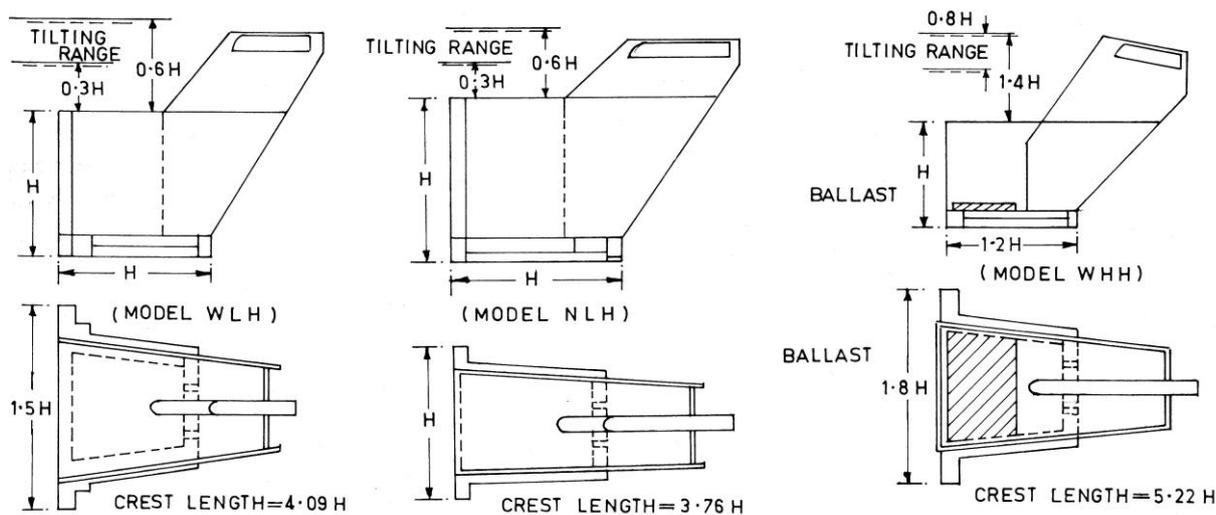


Figure 3-78: Standard configurations of fuse gates. (Falvey et al. 1995)

rehabilitation, for increasing the reservoir storage and also for upgrading spillway’s discharge

**Terminus dam;** (USACE); Lake Kaweah, California, USA. Embankment dam, 78 m high, built in 1962. Purpose of reservoir: Water conservation and flood control. Service labyrinth spillway was added to rehabilitate existing lineal free overflow spillway and to increase reservoir storage (1984). Reservoir capacity:  $0.23 \text{ km}^3$  (see Figure 3-81 and 3-83).

**Case of fuse gate functioning:** Dhatarwadi Dam, Gujarat, India (Figure 3-82). “This is a 35 year old embankment dam (25 m. high) built to provide irrigation benefits. In 2001 its storage was increased by 30%. For this purpose a new spillway was constructed over an ungated spillway (after dismantling of some height as required) consisting of 66 units of metallic fuse gates grouped in 28

sequences for fusing. On June 29-2005, there was a flash flood event, produced by a heavy rain. Flood waters overflowed over the fuse gate spillway and 28 gates fused/tilted as per the designed water level in the reservoir. According to hydrological data, rainfall depth on that date was 235 mm (being maximum during its operational life till that

date); this figure represents 50% of annual precipitation. As mentioned, the event was referred to as a flash flood with sudden increase in reservoir level. Consequences were temporary loss of irrigation area”. (Summary from an investigation study by N. Solanki, Shantilal Shah Engineering College, Bhavnagar, India).



Concrete gravity dam, 32 meters high, with 92 years in operation. Service spillway was rehabilitated for increasing storage and for hydraulic safety with steel fuse gates (Hydro plus, at Erbisti). Detail shows a steel structure and well component on top.

Figure 3-79: Shongweni Dam (South Africa).



Figure 3-80: Typical fusegate installation (HydroPlus)



Figure 3-81: Terminus Dam (California, USA).

Service spillway, 6 units of fuse gate, highest unit used at the world. Arrangement, construction and details of units (USACE, 2004).



Spillway after tilting of 28 out of 66 fuse gates. Flood passing over the spillway -Storm of June 29/2005. (N. Solanki, SSEC).

Figure 3-82: Dhatarwadi Dam, Amreli, Gujarat, India.

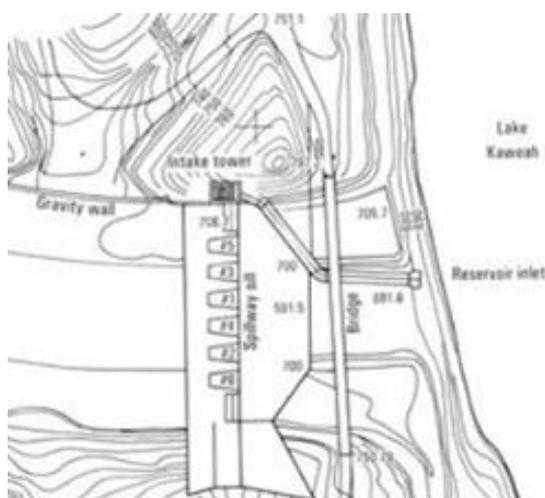


Figure 3-83: Terminus Dam (USA).

Plan view. Service spillway: 6 fuse gates; numbers indicate sequence of tilting, first FG tips at 700 years flood and all FG are washed away with PMF (USACE)

From the point of view of security of dam and reservoir, this fuse gate spillway served its purpose and provided necessary protection by avoiding potential overtopping /dam incident/dam break. As Solanki mentions, operation was as predicted, during this heavy flash flood; increased spillway capacity saved the earth dam and downstream villages, lives, properties, cattle

and farming. However, this “early functioning” (only four years after installation) alerts about hydrological safety of dams located at in the region (Saurashtra) of the country with particular rainfall regime. Solanki refers of two more reservoirs in Gujarat whose fuse gates spillways have functioned.

<p><b>Plan view and arrangement of fuse gates</b></p>	<p>Concrete or steel units in a trapezoidal layout with rectilinear axis, and limited height.</p>	
<p><b>Use</b></p>	<ul style="list-style-type: none"> <li>• Auxiliary or Emergency spillway</li> <li>• Typical use for rehabilitation of existing spillways with inadequate capacity.</li> </ul>	

	<ul style="list-style-type: none"> <li>• For any type of dam: Concrete, Rock fill, Earth fill or Combination.</li> </ul>
<b>Location</b>	<ul style="list-style-type: none"> <li>• Abutment of the dam</li> <li>• On the dam, for Gravity or Arch dams</li> <li>• Over existing spillway to be rehabilitated</li> </ul>
<b>Discharge Capacity</b>	<ul style="list-style-type: none"> <li>• Usually up to 3,000 m<sup>3</sup>/s; however, they have been used for much larger capacity also viz. 30,000 m<sup>3</sup>/s</li> <li>• Unit discharge up to 100 m<sup>3</sup>/s/m</li> <li>• Its capacity increases from initial behavior as a labyrinth weir to a broad crest weir</li> <li>• Design Flood: IDF or greater according to Dam Safety</li> <li>• Capacity can drop due to obstruction by floating debris</li> </ul>
<b>Comments about design and functioning</b>	<ul style="list-style-type: none"> <li>• Foundation: Same as ogee and chute spillway or concrete body of existing spillway or dam</li> <li>• Fuse gates are commercially available in three standard configurations and gate height. These are designated by the width - W (wide) and N (narrow) - and tilting range as being low head (LH) or high head (HH). The standard heights are: 1.5, 1.8, 2.15, 2.60, 3.10, 3.75, 4.5, 5.4, and 6.5 meters. The three types are: WLH, NLH, and WHH (Figure 3-79).</li> <li>• Tilting elevation range: Varies with model see Figure 3-79</li> <li>• Fuse gate is hydraulically more efficient than PK weir.</li> </ul>
<b>Comments about construction</b>	<ul style="list-style-type: none"> <li>• Quality specifications for fabrication.</li> <li>• It is a trademark of Hydro plus.</li> <li>• Several advantages of the structure: (a) Small footprint area so it is easier to install both for new and for rehabilitation of spillways, (b) As units are fabricated separately and placed on site, so construction is easier than labyrinth weir.</li> </ul>
<b>Highlights of this type of work</b>	<ul style="list-style-type: none"> <li>• As a gate, it is very reliable since there are no mechanical or electrical component, and no human operation is needed. It is a robust structure that functions by itself.</li> <li>• Each fuse gate unit is set to overturn in sequence as per reservoir level; all units will fuse at maximum water level.</li> <li>• Selection of design flood: “Fusing flood” is related with frequency of functioning; in some cases, frequency of flood could be modified by regional climate and climate changes. In India, some regions are subjected to special rainfall pattern with heavy short duration’s rains with rapid response (flash floods with high water volume), especially in mountain catchments.</li> <li>• The sequence of fusing should be clearly established for optimal functioning of spillway</li> <li>• Attractive solution for new spillway with ample range of capacity</li> <li>• Environmental impact should be considered when used as emergency spillway far away from dam: Downstream channel capacity, bank erosion, existing facilities, potential damages to residents, and costs.</li> <li>• Extensive research available on classical fuse gates about functioning, clogging, submergence, and other aspects, so it had been said that: “no further research is needed”.</li> <li>• There are recent improved designs of fuse gates by Hydro plus.</li> <li>• Application: To rehabilitate ogee crest and chute spillways.</li> <li>• In use for more about 30 years</li> <li>• Extremely predictable tipping as a function of reservoir elevation which enhances hydraulic safety.</li> <li>• After functioning, a part of storage of reservoir is lost until units are replaced</li> </ul>

	<ul style="list-style-type: none"> <li>• For concrete units, maintenance and surveillance are minimum during its operational life.</li> </ul>
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Table 3-12: Fuse Gates aspects

### 5. Fuse plug (Concrete Blocks)

The concept of a fuse plug was presented earlier for a small earth dam which breaches and fails once it is overtopped. Another concept is based on use of concrete blocks which tilt when water level reaches a fixed elevation and hydraulic forces moves blocks downstream so increasing the open area for flow. This solution was proposed by Hydro Coop (France) and extensive research has been conducted at University of Bistra (Argelia).

The blocks are massive prefabricated units of concrete, with designed dimensions and placed at site (side by side) according to a lineal chain layout. This fuse plug can be used alone as emergency spillway or in combination with other type of weir (labyrinth) as an auxiliary spillway. An advantage over earth fuse plug, besides easiness of construction, is its guarantee of tilting when overtopped by specific extreme floods.

Design of concrete blocks consists of working out their dimensions, checking for their hydraulic functioning, stability and working out the cost; blocks with higher weight will tilt at greater reservoir levels; after tilting the discharge in their case may be about eight times of that before tilting. Optimal block's dimensions commonly increases it by about five fold. Blocks are free-standing and are washed out in progressive fashion (according to water elevation) so the discharge capacity increases by steps; for this planned behavior, the weight of each element varies (with thickness) but the height (elevation of top) is kept the same along the entire length of the fuse plug. The tilting water depth can be several times greater than the block's height.

Flow over fuse blocks corresponds to a broad crest weir that functions according to water elevation and state of the element: (a) Before tilting, with water elevation above blocks but less than tilting head, it is a broad crest weir; (b) After tilting, unit is moved and water tips through that recess corresponding to a flat horizontal sill acting as a short/long and broad weir, according to the approach head over floor; however, this second flow condition is complex to define; so physical models are used. For improving approach flow and nappe, an intermediate streamlined wall is placed between blocks.

Discharge's curve, for flow as initial broad crest weir (Un-tilted blocks), has the expression and notation (for SI units):

$$Q = C * L_t * \left( H_1 + \frac{v^2}{2g} \right)^{3/2}$$

or  $Q = C * B * H^{3/2}$

or  $q = C * H^{3/2}$

Q = Discharge (m<sup>3</sup>/s) and q = unit discharge (m<sup>3</sup>/s per meter of weir or m<sup>2</sup>/s)

L<sub>t</sub> = Length of weir perpendicular to flow (m)

C = Discharge coefficient (m<sup>1/2</sup>/s). It varies during failure process (1.71 to 1.44)

H = Energy upstream of weir section = H<sub>1</sub> + V<sub>1</sub><sup>2</sup>/2g (m)

This expression can be simplified assuming free flow at critical depth on the weir and flow approaching as uniform with low velocity, as:

$$Q = C * L * h^{3/2}$$

here discharge coefficient C varies around 1.705

h = Upstream head (m) = Elevation of reservoir – Elevation of weir crest.

See Figures 3-84 to 3-89. Table 3-13 presents a summary of aspects related to use, design, construction and control of concrete fuse plugs for new and rehabilitation of existing spillways.

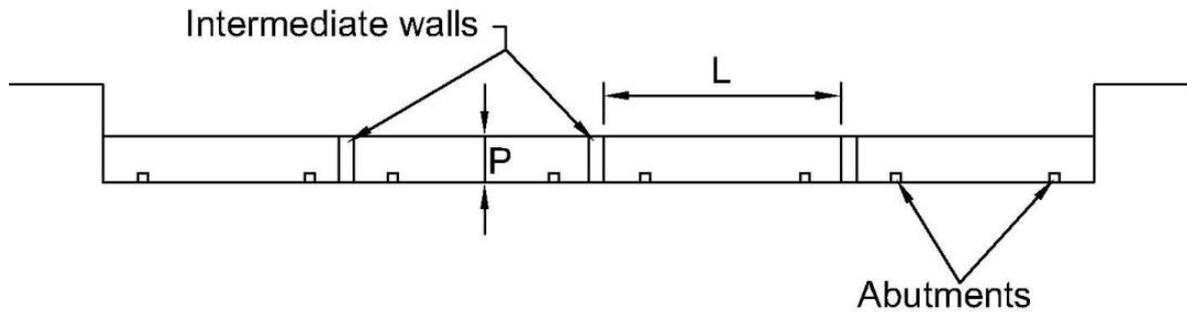


Figure 3-84: Downstream view of concrete fuse plug (HydroCoop).

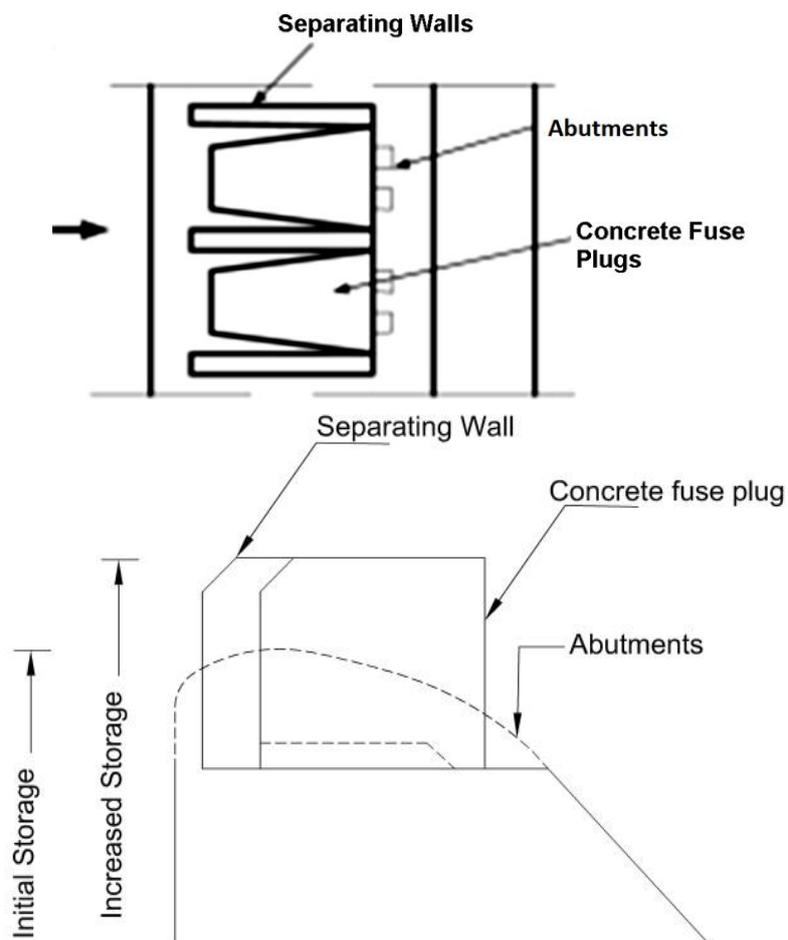
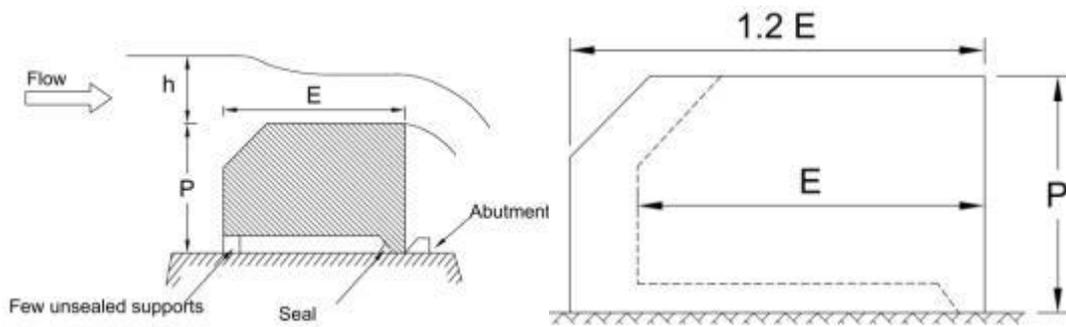


Figure 3-85: Arrangement of concrete fuse plug used to increase reservoir storage capacity (Khatsuria, 2005).



Typical details for supporting and sealing. Intermediate wall between elements with an extended length of 0.2E (Hydro Coop)

Figure 3-86: Fuse block with height



<b>N°FUSEPLUGS</b>	<b>1</b>	<b>2</b>	<b>3</b>	<b>4</b>	<b>5</b>	<b>6</b>	<b>7</b>
<b>LENGTH (m)</b>	9.62	9.63	9.62	9.62	9.63	9.63	9.60
<b>HEIGHT (cm)</b>	50	50	50	50	50	50	50
<b>THICKNESS (cm)</b>	78	82	85	89	94	98	103
<b>WATER HELD ABOVE THE BLOCK FOR TILTING (cm)</b>	66	70	74	78	82	86	90
<b>FLOW FOR TILTING</b>	83	106	132	158	184	210	237

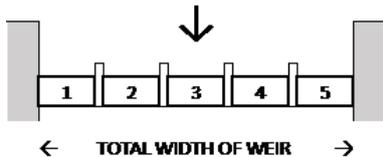
Figure 3-87 Wedbila Dam, Burkina Faso. Spillway with 7 Fuse plug concrete blocks and their technical features ( Hydro Coop, 2013).



Figure 3-88: Wedbila Dam, Burkina Faso. Small embankment dam. Spillway with fuse plug con-crete blocks (HydroCoop).



Figure 3-89: Physical model of concrete fuse plug alone and in combination with PK weir (University of Bistra).

<b>Plan view and arrangement</b>	Concrete elements placed in a rectilinear layout, side by side, with limited height.	
<b>Use</b>	<ul style="list-style-type: none"> <li>• Auxiliary or Emergency spillway</li> <li>• Typical use for rehabilitation of existing spillways with inadequate capacity as modification of existing spillway or as a new additional spillway.</li> <li>• For any type of dam: Concrete, Rock fill, Earth fill or Combination.</li> <li>• As a fuse plug it operates once during dam life time, then it has to be re installed.</li> </ul>	
<b>Location</b>	<ul style="list-style-type: none"> <li>• Abutment of the dam</li> <li>• At a saddle along reservoir rim</li> <li>• Over existing spillway to be rehabilitated</li> </ul>	
<b>Discharge Capacity</b>	<ul style="list-style-type: none"> <li>• Usually recommended up to several thousand m<sup>3</sup>/s</li> <li>• Discharge capacity increases progressively according to a predefined sequence of block's tilting</li> <li>• As auxiliary spillway, usually selected IDF (normally the defined reservoir level before tilting) is 100 years. For emergency spillway design flood can be greater according to Dam Safety</li> <li>• Can be used to increase storage in reservoir</li> </ul>	
<b>Comments about design and functioning</b>	<ul style="list-style-type: none"> <li>• Foundation: Same as ogee and chute spillway or concrete body of existing spillway</li> <li>• Concrete block has an optimal height based on hydraulic and economic study, common <math>P = 1.5h</math> (shape of block and notations as previous figure)</li> <li>• Length of block <math>L</math> is greater than <math>E</math>, and <math>E</math> is greater than <math>P</math>.</li> <li>• Blocks are placed with narrow intermediate walls to improve approach flow condition.</li> <li>• Minimum number of blocks is 4 or 5 (as recommended by manufacturer)</li> <li>• Key design is establishing the required <math>E/h</math> ratio at tilting water elevation, for preliminary dimensions: <math>h = E - 0.4P</math></li> <li>• Length <math>[L]</math> of block as designed usually equal for all blocks.</li> <li>• Physical model investigations are.</li> <li>• Crest level at Normal Operation Pool Level; it could be higher to increase reservoir volume</li> <li>• Obstruction by floating debris is not a significant problem since height of intermediate walls is equal to blocks, so there is not interference which favors accumulation of long debris.</li> <li>• Efficiency is less than labyrinth or PK weirs.</li> </ul>	
<b>Comments about construction</b>	<ul style="list-style-type: none"> <li>• Quality specifications for fabrication. Key parameter is concrete density.</li> <li>• Several advantages of the structure: (a) Small footprint area so it is easier to install at new or rehabilitated spillways, (b) Units are pre-fabricated and placed on site so construction is easier than cast on site structures.</li> <li>• When used for rehabilitation of existing spillway, demolition work is less than labyrinth or PK weirs.</li> </ul>	
<b>Highlights of this type of work</b>	<ul style="list-style-type: none"> <li>• Weight of each block unit is set to overturn at progressively higher reservoir elevations, so not all units are lost for floods lower than that used in designs; in that extreme event all the fuse plugs will tilt and will be moved away.</li> <li>• Attractive solution for new emergency spillway with ample range of capacity</li> </ul>	

	<ul style="list-style-type: none"> <li>• For a new spillway, fuse blocks involve about half the concrete of a conventional rounded crest.</li> <li>• Environmental impact should be considered when it is located, as emergency spillway, far away from dam: downstream channel capacity, bank erosion, existing facilities, potential damages to residents, and costs.</li> <li>• Its hydraulic functioning is simple and accurate.</li> <li>• Important structural details are: placing downstream abutments to avoid sliding of block, sealing of contacts and reducing friction between blocks or intermediate walls.</li> <li>• It is important to check freeboard allowance for the dam for water levels needed for fuse plug operation. When used for heightening an existing spillway, “P” of blocks is limited to ¼ of vertical distance between dam’s crest and sill’s crest.</li> <li>• Extremely predictable tipping as a function of reservoir elevation which is not the case for conventional fuse plug (earth dike)</li> <li>• After functioning, a part of storage of reservoir is lost until units are replaced. Temporary flash board could be used.</li> <li>• Fuse blocks can be used in combination with PK weir for a cost-effective solution.</li> </ul>
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Table 3-13: Concrete Fuse Plug aspects

## 6. Overtopping of dam with protected section

Although some overtopping over a concrete /masonry dam may not lead to dam failure, such an overtopping may be disastrous in case of an embankment dam.

In some dams it is not possible to provide an additional, auxiliary or emergency, spillway because of non-availability of a suitable site, unfavorable geology or physical limitations of the existing spillway. An alternative, is to protect a portion or total length of dam and allowing overflow - with little or no damages during IDF and avoiding dam failure/break. This solution applies to both concrete dams and embankment dams (which are much more vulnerable to failure if overtopped). The protected zone acts as an overflowing weir with low head and low unit discharge.

Protection can be provided with several materials, some flexible and other rigid. Among first group are gabions, riprap, geosynthetics products (cells, membranes, mats, others) etc. Rigid solutions include use of soil cement mixtures, rolled compacted concrete (RCC), precast armoring concrete blocks (ACB) and reinforced concrete (CVC).

Selection of the type of protection depends on the type of dam and its height, potential hazard assigned to the dam, expected discharge, behavior of protection material and its durability and cost of possible options.

As in this case, the aim is to allow overtopping over the embankment dam, a sound analysis of failure modes is required to guarantee the safety of the dam. This Manual is focused, mainly to dams with a height larger than 15 meters with significant and high potential of hazard if dam breaks. Thus, the protective covers are required to be robust enough, reliable and less maintenance dependent.

During the last decade, protection of embankment dams with an RCC overlay on the crest and downstream slope, has become an attractive option, being the most used system or material. Roller compacted concrete protection is carried out in lifts over the sloped surface with steps, so it works as a steeped spillway which provide substantial energy dissipation. The main advantages of RCC are its suitability to flow with significant velocities on downstream slope, its resistance to erosion or abrasion processes, it is resistant to debris impact and

vehicles loads on crest and good hydraulic performance with supercritical flow. In addition, construction with RCC is these days becoming more and more popular because of its inherent advantages of rapid and economical construction. RCC is being

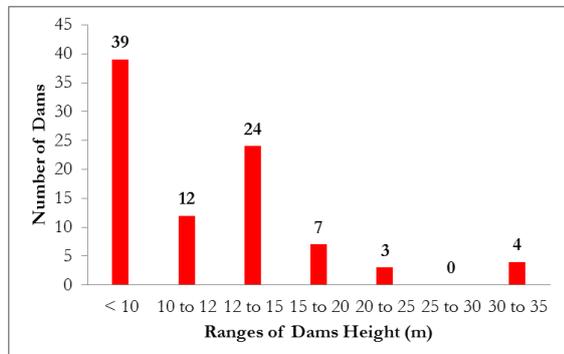


Figure 3-90: Embankment Dams with RCC Protection (Hansen at FEMA 2014).

used both for the construction of new dams and as well as for rehabilitation of old dams.

As stated by FEMA (2014): *“The development of RCC technology has provided a successful method of erosion protection of embankment dams, which has proven to be cost effective while affording a number of other advantages. RCC construction is normally very rapid compared to conventional concrete construction, with minimal project disruption. In most cases, construction for overtopping protection is limited to the dam crest and downstream slope, with little to no impact to reservoir operations. Depending upon the site conditions and discharge requirements, the entire length of the embankment dam can be used by armoring the crest and downstream face with RCC, or a selected portion of the embankment crest can be lowered for use as an RCC-lined spillway. However, lowering the embankment crest can potentially change the downstream flood risks and potential liabilities, and this lowering should be evaluated for each project”.*

FEMA presents Hansen’s list of dams (USA) protected with RCC (119 cases since 1984) within this range of conditions: (1) Dam’s height: 4 to 35 meters, unit discharge: 0.8 to 13.3 m<sup>3</sup>/s/m (average of 7.5 m<sup>3</sup>/m/s), water depth on crest: 0.4 to 4.6 m. Figure 3-90 shows number of embankment dams corresponding to different ranges of dam’s

height with RCC protection (FEMA, 89 cases with data). Schnabel Engineering (USA) reports 14 embankment dams protected with a RCC cover, from 1998 to 2017, with dam’s heights in the range 10 to 24 meters, and with overtopping water depths between 1 and 2.8 m.

Discharge corresponds to free overflow, being the dam’s crest a broad weir. The hydraulic functioning of Stepped Spillway is presented later in this chapter of the manual.

Figure 3-91. shows typical flow conditions over an overflowing type of an embankment dam such as velocity on downstream slope, need to protect the dam’s toe, need to incorporate some terminal structure to cope with excess flow’s energy and erosion, and possibility of hydraulic jump on the slope according to tail water elevation.

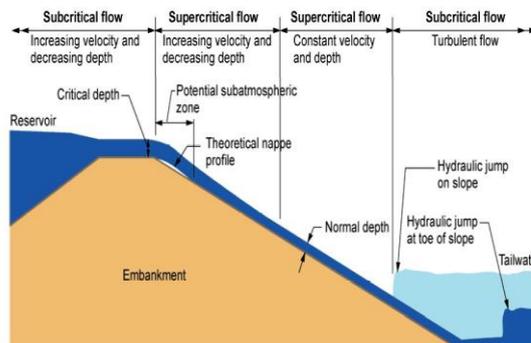


Figure 3-91: Typical hydraulic conditions during embankment’s overtopping (USBR, FEMA).

Downstream slopes of embankment dams are generally 2:1 to 3:1 (H: V); on these slopes, RCC can be placed in lifts, with steps of 0.3 to 0.6 m height, which improves energy dissipation. The minimum thickness of RCC’s overlay is defined according to construction method, width of layer is commonly 3 m. due to typical compaction equipment.

Additional components to guarantee hydraulic functioning, erosion control and stability (seepage, uplift and sliding) include terminal works (apron, protected basin, energy dissipater), cutoff walls (upstream at crest and downstream at toe, training walls

along the RCC or conventional reinforced concrete, abutment protection (as needed), drains, and other geotechnical components. Figure 3-96 to 3-97 show typical RCC cover and example of this type of solution for dam's overtopping.

In relation to performance/functioning during extreme events of flood and factual experiences, there are few cases reported around the world. In USA, few dams have experienced significant flows and for long durations. Based on limited experience, embankments with RCC overtopping protection have performed well during overtopping, with only minor damages. Several cases (cited by Abdo and Adaska, in FEMA) have performed well with overtopping depths of up to 3 meters, with damage limited to surface erosion and minor spalling.

Hansen and Fitzgerald have reported four cases of protected dams functioning with high frequency floods and seven with rare events (floods greater than 100-year return period), in all cases (with high unit discharges), performance was reported as excellent, with minimum erosion and instability.

RCC is also used to protect concrete gravity dams; in this case the downstream face of the dam, abutment and downstream foundation are required to be protected. Geological characteristics of dam site are the key for designing the rehabilitation and its coverage, to prevent scour and dam's undermining when it overflows. Common solution is basically a massive protection or buttress of RCC against dam and abutments (with or without reinforcement).

As compared to low height embankment dams, much higher concrete dams can be considered for this kind of protection, if required, from hydrological/hydraulic considerations.

### Examples of overtopping protection with RCC

- a) RCC protection in dam's rehabilitation, for increasing the dam safety

Several embankment dams along Yellow river (NRCS), Georgia, USA were rehabilitated to upgrade the dam to perform as an auxiliary spillway. Potential dam hazard was raised from low to high due to changes in downstream consequences in case of eventual dam failure as well as due to changes in purpose of the reservoir.

Figure 3-92 and 3-93 show an embankment's dam during overtopping and after flood passing, the terminal structure in this case is a stilling basin (USBR type) (Hansen and Fitzgerald). Figures 3-94 to 3-99 shows examples of rehabilitation of dams of different heights.

For RCC design and geotechnical, structural and construction criteria, ample information can be found in publication elaborated by USBR for FEMA (2014): "Technical Manual: Overtopping Protection for Dams"



Figure 3-92: RCC overtopping protection. Left Hand Valley Dam (13.7 m high), Colorado, USA.

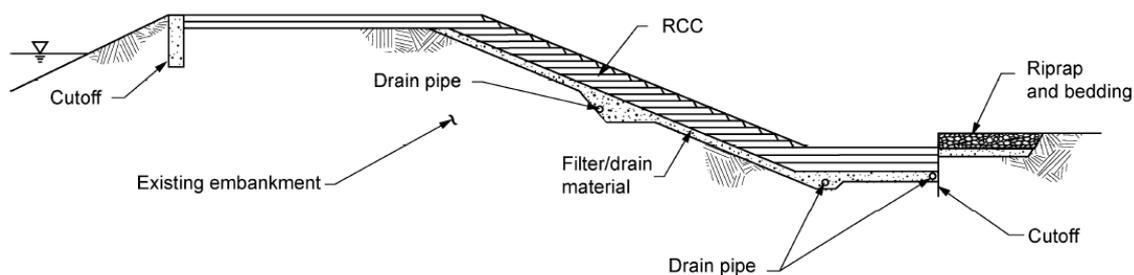


Figure 3-93: Typical Section: RCC overtopping protection (PCA at FEMA).



Y15 Dam (9.2 m high), Georgia, USA. IDF was  $\frac{1}{2}$  PMF. During flood, water depth on crest 0.37 m and unit discharge of  $0.4 \text{ m}^3/\text{s}/\text{m}$ . Soil and vegetation cover was locally washed out, but RCC was not damaged (Hansen and Fitzgerald.).

Figure 3-94: RCC overtopping protection. Yellow river.



Yellow river, Y14 Dam (12 m high), Georgia, USA. IDF was PMF. During flood, water depth on crest 0.60 m (Hansen and Fitzgerald.).

Figure 3-95: RCC overtopping protection.



On a 3:1 slope. Yellow river, Y16 Dam (10.4 m high), Georgia, USA. IDF was  $\frac{1}{2}$  PMF (Hepler et alia)

Figure 3-96: RCC overtopping protection t entire length of dam.



Figure 3-97: Thomaston Dam, Georgia, USA. RCC overtopping protection in entire length of dam.



Camp Dyer Diversion Dam (Masonry dam, 23 m high, built 1926), Arizona, USA. Rehabilitated for dam safety in 1992.

Figure 3-98: RCC overtopping protection, buttress downstream (USBR, FEMA).

## 7. Unlined channels

Other solutions for spillways, especially for emergency function, are channels excavated (with and without protection) in a saddle of the reservoir rim. Selection of this option depends on: topography, geology, inflow discharge flood (IDF), short- and long-term durability, need of maintenance, hydraulic behavior, downstream consequences when functioning and cost. The key factor to be considered in design is to avoid erosion and head cutting potential.

The section of this channel is open without any type of closure such as any plug; sometimes there are some control components such as concrete sills with walls followed by local riprap or concrete blocks on both sides. This solution is easy and a low-cost solution when compared with other solutions for emergency spillways. Under DRIP this solution has been adopted to augment the spillway capacity in Marhi dam, Jirbhar dam, Umrar dam and Kankarkhera dams in Madhya Pradesh. Such spillways are also called as flush bars.

The channel's discharge capacity can be worked out as a broad crested weir if there a concrete sill control or simply by using Manning equation. Flow profile can be determined according to slope and



Santa Cruz Dam (Concrete gravity dam, 46 m high, built 1929). New Mexico, USA. Rehabilitated for dam safety in 1990.

Figure 3-99: RCC overtopping protection, buttress downstream entire crest length and on abutments (USBR, FEMA).

in channel section and lack of maintenance are also some aspects to consider.

It is also important to establish erodibility of the channel especially when the rock is weathered/highly jointed. There are several soft tools to perform hydraulic analysis (USACE, NRCS, other). For example, REMR (erosion prediction procedure) classifies soil erosion potential as: high, significant, moderate and slight erosion risk. Typical ranges of the parameters as reported for the functioning of these channels are longitudinal slope 2 to 24 degrees and flow velocity 1.2 to 4.6 m/s.

For spillway's channels excavated in poor rock, there are other approaches to estimate erosion that take into account the mechanisms involved in dislodgement and movement of fragments. Two procedures, based in characteristics of rock such as type/condition of rock, compressive strength, orientation of joints, rock quality designation (RQD), structure and other properties, were proposed by Annandale Van Schalkwyk ( Khatsuria, 2005). Those approaches can be used to assess possibility and extent of erosion due to action of flow over the rock surface, so they may be helpful while deciding about feasibility of an unlined spillway channel.

This approach can also be used to evaluate making a decision on accepting an unlined rock channel downstream of spillway's control structure or after a short concrete chute. (See Figure 3-100 and 3-101).

If unlined channel spillway is found to be unsafe based on velocities and erosion considerations, lined sections can be considered according to the type of base material (soil or rock) and flow conditions. In this case, for economic and environmental reasons, materials for lining should be natural, easy to find and to place. Design of types of protection are based on hydraulic and geotechnical criteria. Under the DRIP Project, fall structures have been provided with energy dissipation arrangements in some projects in view of scour taking place due to flow over weathered /highly jointed rock in the spill channel d/s of the main spillway structure and extending towards the earth dam. Examples are Sanamachhakandana and Damsal dams in Odisha. Also, in some cases where the width of spill channels was less than the spillway width, they have been widened as per site-specific requirements. Obstructions to flow were removed where necessary.

Emergency spillways with unlined channels are maintenance's dependent so in order to guarantee their functioning, frequent inspections and upgrading actions should be included in the dam's maintenance program. Basic activities are related to obstruction due to natural vegetation, trees, bushes, bank instability, erosion, rock slide, weathered rocks, debris accumulation, other obstructions to flow and any other situation that can affect channel capacity or its stability.

### 3.2.2 Conveyance feature

The flood waters or spills from the spillway are passed from the reservoir at a high elevation to the river downstream of the dam, at a lower elevation through its conveyance structure.



Robert Bourassa Dam, Quebec, Canada. Stepped profile, 1 km long, 122 m wide, Capacity: 16,250 m<sup>3</sup>/s.

Figure 3-100: Unlined chute excavated in rock (K. Warren Frizell).



Oroville Dam, California, USA, 2017 (Aerial photo, Daily Hampshire Gazette).

Figure 3-101: Rock erosion at natural slope downstream of emergency spillway after operation.

The conveyance structure can be located either on surface or underground.

In the first case it can be either the d/s of glacis or over fall spillway or the chute of a chute spillway located on an abutment of the dam. In case of spillways located far away from the dam, as sometimes in auxiliary or emergency spillways is the case, this component could be present or not according to local conditions (topography and geology).

In the second case, tunnels are located through an abutment in case of tunnel or morning glory spillways; also, there is another conveyance structure, a conduit, which may be provided in case of a conduit spillway.

From the hydraulic point of view, some basic aspects in respect of conveyance structures can be identified as follows:

- The flow on the glacis of over fall spillways or on surface channels (chutes) of chute spillways and even in tunnels is generally open channel flow. Pressurized flow can be, in some cases, accepted in tunnel spillways but due to issues like cavitation and other, it is usually avoided.
- Steep slopes due to height of the dam and/or topographical conditions from reservoir to the river.
- Discharging capacity of the conveyance structure must remain equal to the discharge from the control section upstream.
- The flow in the conveyance structure is generally supercritical flow with the following specific properties: high velocity, turbulence, aeration of water mass, cavitation's potential, shockwaves generation, changes in water elevation, flow singularities, local high pressures and high energy content. These aspects become hydraulic loads to be considered in design and rehabilitation of the chute or tunnel.
- Since chutes are located downstream of the reservoir, the hydraulic loads act mostly on account of seepage. Water seeps through the rock mass and foundation and exert uplift on the conveyance structure with possibility of erosion which can manifest as piping and undermining. Measures to take care of uplift include provision of drainage system, anchors, etc.
- The height of chute walls must be enough for bulked water depth, waves and splashing.

### 3.2.3 Stepped chute spillways

Stepped spillways correspond to a free overflow discharge with a control structure which spills into a chute with stepped

bottom. Since 35 years this has been adopted as a very convenient solution for gravity rolled concrete dams (RCC), due to constructive reasons and cost. The evolution towards higher RCC dams led to much research on the hydraulics of this type of spillway, which is still under study. Currently, this solution is accepted as:

- Service spillway for RCC dams, built directly on the downstream face of the dam.
- Auxiliary or emergency spillways, non-conventional type, basically in two cases: :
  - Earth dams with a controlled overtopping section
  - Stepped chutes at any site along the reservoir rim, made by gabions, masonry, concrete or other material.

In India, stepped chutes have been used in small dams, mainly built of masonry. RCC in dams was first used in 2001; currently (2021), the dam register (CWC, 2019) indicates there are a few dams with this material and not all with the spillway on the dam's body or stepped type. Two RCC's dams with stepped spillways are reported at the Ghatghar Project in Maharashtra: Ghatghar Upper and Lower Dams (15 and 86 m high respectively). Figure 3-102 shows Lower Ghatghar Dam stepped spillway, designed for  $192 \text{ m}^3/\text{s}$  (specific flow =  $2.7 \text{ m}^3/\text{s}/\text{m}$ ).



Figure 3-102: Ghatghar Lower Dam. First RCC dam in India; 14 years in operation (Patel Engineering)

Around the world, RCC dams with stepped chutes have improved in hydraulic performance due to continuous research and with the benefit of investigations through diverse physical models that have contributed to the design of these structures. In the case of emergency spillways, embankment dams with overtopping has gained interest and is increasingly used as a safe option, in particular, in dams less than 20 meters high. Figure 3-103 shows an example of this non-conventional auxiliary's spillway.



Figure 3-103: Tongue River Dam, USA. Two spillways: service (right) and auxiliary stepped chute (left) (FEMA, 2014)

Among the advantages of this type of spillway, as mentioned by several researchers, are:

- High energy dissipation along the channel with reduced residual energy at the toe of chute in comparison to the conventional smooth channel, which implies smaller energy dissipator.
- Even though main application of stepped chutes is for uncontrolled conventional overflow spillway, there are examples with controlled spillway and with non-conventional control as PKW.
- The steps allow aeration of the flow, which reduces the occurrence of cavitation.
- Research has shown that, for high dams (RCC), the behavior of this spillway is reliable with increasingly large specific flow rate (greater than  $100 \text{ m}^3/\text{s}/\text{m}$ ), in relation to commonly used limit of 25

$\text{m}^3/\text{s}/\text{m}$ ; however, for overtopping on embankment dams this figure has been safely kept less than 7.5 to  $10 \text{ m}^3/\text{s}/\text{m}$ .

- It is a solution easily adaptable to the downstream face of RCC concrete dams, taking advantage of the steps resulting from the construction process.
- It is a convenient option for overtopping section on earth dams due to location and its construction.
- For embankment dams several materials with adequate abrasion and debris impact resistance can be used: gabions, precast concrete blocks and rolled compacted concrete, even geotechnical products for low dams.

The following is a summary of functioning of stepped chutes, focused in a hydraulic evaluation; the reader may also refer to the extensive research available in key references of hydraulic institutions from Australia, Japan, United States, Portugal and others countries, included in this Manual.

Like other overflow spillways, the control section is a sill with an ogee or WES geometry that defines the specific discharge flow ( $\text{m}^3/\text{s}/\text{m}$ ) according to the expression:  $q = CH^{3/2}$ , where "C" is the discharge coefficient (in SI units) and "H" is the hydraulic head (m) on the crest. Flow over stepped chutes is limited to convenient values of "q" and "H". See Figure 3-104 to 3-109. The flow over the chute, a water-air mix, is verified in two types (according to "q") and clearly defined and separated by a transition flow, as shown in Figure 3-106:

- a) Nappe (NA): Jet drops and impinges the step, followed by a partial or total hydraulic jump, there are air pockets on the steps.
- b) Skimming flow (SK): Stream skimming over the steps and stable vortices develop on steps defining a pseudo bottom between edges.

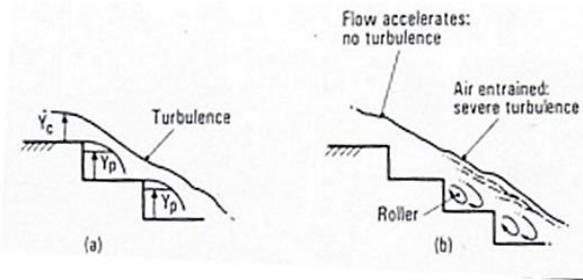


Figure 3-104: Types of flow in a steeped chute (Chanson, 1994)

Typically, for design discharge (IDF), flow corresponds to SK or type "b"; nappe flow can occur with low discharges and for low slope as some cases of overflow embankment dams. Figure 3-104 shows SK flow over a hydraulic model. Figure 3-106 summarizes the data of ample research and predicts the type of flow for a rectangular section (critical depth;  $d_c = [q^2/g]^{1/3}$ ), a given flow rate and a geometry of the steps ( $h$  = height and  $l$  = tread length). In the figure, "TRA" corresponds to the transition zone, which should be avoided in the design.



Figure 3-105: Skimming flow over 0.30 m high step with  $q = 1.4 \text{ m}^3/\text{s}/\text{m}$  (CSU, USBR)

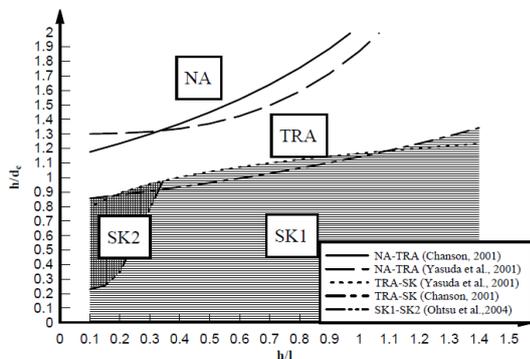


Figure 3-106: Prediction of flow regime in steeped spillways (Chanson, 2001 and Ohtsu, 2004)

Figure 3-107 defines the two regions of SK flow along a long steeped chute over a high dam (RCC); for shorter chutes, flow does not reach the equilibrium state (as in small RCC dams or some cases of embankment dam overtopping). The first region begins at crest with a smooth clear water surface and the second is a water-air mix (three sectors) that reaches a fully aerated uniform flow. Figure 3-108 shows a detail of SK flow and nomenclature used. Both figures allow to define aspects related to the hydraulic functioning of the spillway, such as:

- Non-aerated flow
- Section where the aerated flow starts (point of interception)
- Flow length up to the uniform flow
- Water depth in aerated flow ( $Y_{0.9}$  or  $Y_{90}$ )
- Energy losses in the slopped chute ( $\Delta H$ ) and residual energy ( $H_{res}$ ) at toe

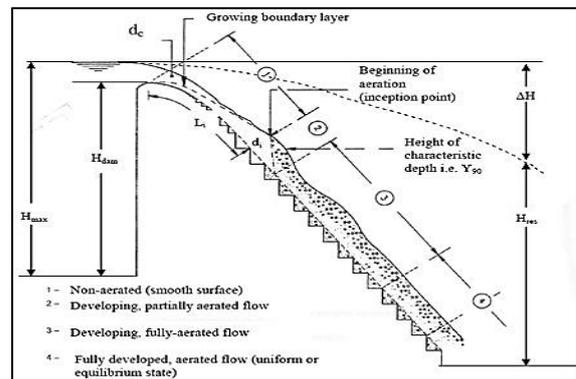


Figure 3-107: Flow regime on steeped spillway (Matos, 1999)

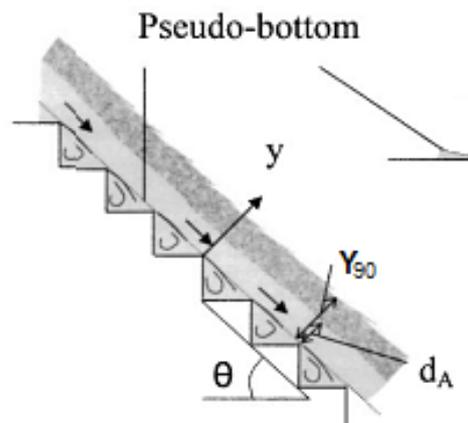


Figure 3-108: Detail of skimming flow for  $19^\circ \leq \theta \leq 55^\circ$  (slopes 2.9:1 to 0.7:1) (Ohtsu; Yasuda and Takahashi, 2004)

In assessing the hydraulic safety of a stepped chute, the hydraulic aspects mentioned below allow to verify:

- Spillway capacity
- Length to reach uniform SK flow
- Height of walls in the stepped channel
- Cavitation potential and its consequent damage to the steps
- Residual energy ( $H_{res}$ ) and thus, operation and dimensions of the energy dissipator or downstream protection

Hereinafter the following terms will be used to indicate and differentiate the types of dams: RCCD will be used for "Rolled Compacted Concrete Dam" and "OED" for "Overflow Embankment Dam".

In relation to the hydraulic operation of an existing stepped spillway, focus of evaluation encompasses:

- Higher updated flow rate than the original design: It is essential to check whether the resulting specific flow rate ( $m^3/s/m$ ) is acceptable; in particular, in the case of OED it is important to keep "q" below the recommended limit and to revise geometry of steps.
- Type of flow: SK is common in RCCD. Both types of flow can occur in OED: NA (for low flow rates) and SK.
- The crest of the weir matches Full Reservoir Level (FRL)
- Profile: In RCCD, the crest's sill is an ogee or WES profile, immediately downstream where the bottom slope is steep, lower-rise steps are used; for higher "q", it is important to check the operation of this zone. In OED, crest is a broad weir with critical depth formation; in this case, it is common to use equal-height steps throughout the chute.
- Aerated flow: once the point of inception is reached, the flow occurs in

two regimes: gradually varied and uniform (or pseudo-uniform). The length of the chute defines whether uniform flow is reached.

- High velocity aerated flow: height of walls and residual energy of flow entering to dissipator.

The basic procedure presented in Table 3-13 is a summary for an initial evaluation of hydraulic performance; this type of spillway has a complex functioning that should be deeply analyzed, in some cases, with the help of physical models. The expressions in Table 3-13 are based on:

- SK flow occurs for IDF discharge
- Length of the steeped chute is sufficient to reach the uniform aerated regime. It is common in high RCCD with steep slope; in small dams and gentle inclinations (2.5: to 4:1), as OED, this condition could not be reached. These cases should be reviewed with specific methodologies.
- Stepped chutes with slopes in a range of 3:1 to 0.7:1, (moderate to steep slopes) so, with the respective indications, it is valid for RCCD and OED.
- Used variables or parameters:
  - $Q$  = discharge ( $m^3/s$ )
  - $W$  = spillway width or spill length (m)
  - $q$  or  $q_w$  = specific discharge of clear water ( $m^3/s/m$ );  $q = Q/W$
  - $h$  = height of step (m) (also "S")
  - $\theta$  = angle of chute or slope
  - $k$  = step roughness height perpendicular to slope (m);  $k = h \cos \theta$

- Fr = Roughness Froude number;  $F_r = \frac{q_w}{\sqrt{g \sin \theta k^3}}$
- $H_{dam}$  = height of dam (m) from spillway crest to chute's toe
- $H_{max}$  = maximum energy (m);  
 $H_{max} = H_{dam} + 1.5d_c$
- $H_{res}$  = residual energy (m) at toe of chute;  
$$H_{res} = d \cos \theta + \alpha \frac{q^2}{2gd^2}$$
- $d$  = clear water depth (m); all "d" measured normal to pseudo bottom of chute
- $d_c$  = critical depth (m);  $d_c = (q^2/g)^{1/3}$
- $d_w$  = representative (or equivalent) clear water depth (m) for uniform flow
- $C_{mean}$  = average air concentration ratio (volume of air per unit volume of air and water)
- $Y_{90}$  = aerated water depth (air concentration 90%) (m)
- $H_w$  = height of lateral wall (m);  
 $H_w = [\text{Safety factor}] Y_{90}$
- $U_w$  = average velocity of flow (m/s)
- $f$  = Darcy-Weisbach friction factor for steeped chute
- $F_w$  = Froude number for representative (equivalent) depth;  
$$F = \frac{U_w}{\sqrt{gd_w}}$$
- $\alpha$  = Kinetic energy correction factor (1.10 to 1.16)

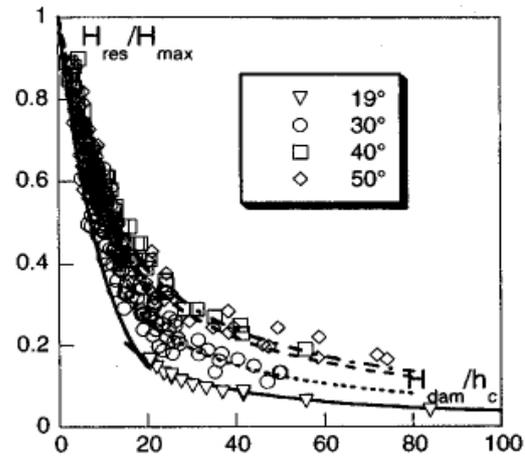


Figure 3-109: Relative residual energy head ratio ( $H_{res}/H_{max}$ ) as a function of relative spillway height, ( $H_{dam}/h_c$ ) (Boes and Hager 2003,  $h_c = d_c$ )

- The energy dissipator at toe of chute can be a flip bucket, stilling basin or horizontal concrete slab. In case of good quality rock foundation, direct discharge can be accepted according to the residual energy content of flow.
- In this type of unconventional spillway, it is important to use recent research information. In particular, in the case of OED, reliable parameters and methodologies should be used to ensure proper operation, stability and durability of the work.
- This type of spillway should be verified in hydraulic models for unusual operating conditions: aeration, cavitation, channel geometry, energy dissipation, among others. Appendix "D" of this Manual contains detailed information on the use and application of hydraulic models.

Some comments to be consider to select steeped chutes for dam rehabilitation:

Figures 3-110 to 3-112 show examples of stepped spillway for RCCD and OED. Figure 3-113 to 3-115 show hydraulic models for specific conditions.



Figure 3-110: Upper Stillwater Dam, Utah, USA (1987)



Figure 3-113: Hydraulic model of stepped spillway with converging wall



Figure 3-111: Construction of steeped spillway on an earth dam; Las Vegas, USA. (USACE and H. Chanson, 2009)



Figure 3-114: Hydraulic 3D model and stepped spillway; precast concrete blocks and trapezoidal section; Barriga Sam (Moran and Toledo, 2006)

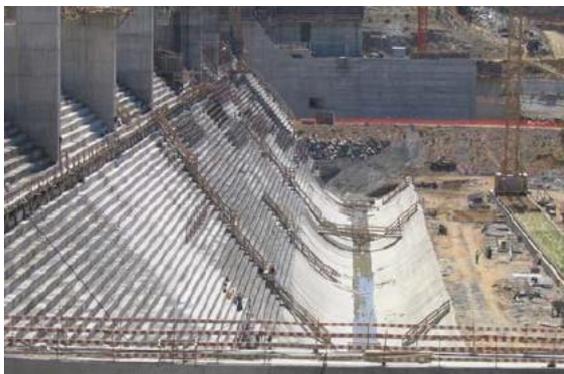


Figure 3-112: Pedrógão dam, Portugal (2005), steeped spillway for  $q = 40 \text{ m}^3/\text{s}/\text{m}$  with flip bucket (EDIA)



Figure 3-115: Hydraulic model stepped (slope 2:1), Stilling basin: USBR Type III (Frizell, et al.; 2016)

Flow condition or flow parameter	Symbol	RCC dams (RCCD)	Overflow embankment dams (OED)
Slope of pseudo bottom (angle $\theta$ )	S	0.5:1 to 0.8:1 (63.4° to 51.3°)	4:1 to 2:1 (14° to 26.6°)
Check for skimming uniform flow	SK	$H_{\text{dam}}/d_c \geq 15$ to 20 (Boes and Hager)	$H_{\text{dam}}/d_c \geq 15$ to 20 (Boes and Hager)
Height of steps	h (m)	$h \geq d_c/3$ (Stephenson) $0.25d_c \leq h \leq 1.17d_c (\tan \theta)^{1/6}$ (Ohtsu et. al.)	$h = 0.30 d_c$ (Tozzi) $0.25d_c \leq h \leq 1.17d_c (\tan \theta)^{1/6}$ (Ohtsu et. al.)
Depth of clear water for uniform flow	$d_w$ (m)	$d_w/d_c = 0.215 (\sin \theta)^{-1/3}$ (Ohtsu with $f=0.08$ )	$d_w/d_c = 0.283 (\sin \theta)^{-1/3}$ (Ohtsu with $f=0.18$ )
Mean air concentration at uniform flow	$C_{\text{mean}}$ (%)	$C_{\text{mean}} = 0.75 (\sin \theta)^{0.75}$ (Hager)	$C_{\text{mean}} = 0.75 (\sin \theta)^{0.75}$ (Hager)
Depth where C is 90% (bulk depth)	$Y_{90}$ (m)	$Y_{90} = d_w/(1-C_{\text{mean}})$ $(Y_{90}/k) = [0.5 (\cos \theta)^{1.5a}/\cos \theta] (\text{Fr})^a$ $a = (0.1 \tan \theta + 0.5)$ (Boes and Hager)	$Y_{90} = d_w/(1-C_{\text{mean}})$ $Y_{90} = 1.75 d_w$ (Ward)
Height of walls	$H_w$ (m)	(1.2 to 1.4) $Y_{90}$ (Ohtsu et al.)	(1.5 to 2) $Y_{90}$ (Boes and Hager; Wood)
Residual energy at toe of the chute	$H_{\text{res}}$ (m)	From Figure 3-109	From Figure 3-109
Clear water velocity at uniform flow	$U_w$ (m/s)	$U_w = q/d_w$ .	$U_w = q/d_w$ .
Froude number at chute's toe	$F_w$	$F = \frac{U_w}{\sqrt{gd_w}}$	$F = \frac{U_w}{\sqrt{gd_w}}$
Cavitation	Natural aeration, commonly, is sufficient to avoid cavitation; however, depending on slope and specific flow rate it is advisable to review the cavitation potential.		

Table 3-13: Summary for initial hydraulic evaluation of stepped chutes

### 3.2.4 Gates for spillways

In controlled spillways, the gates are the mechanical features to control outflow discharges. They have several advantages, especially for handling large discharges. The gates can be installed on the crest of weir; thus, they operate as an orifice for partial opening and as a free weir for full opening; also, gates are installed at the inlet of sluices. They are located at an elevation lower than the normal operation level of reservoir (FRL); so the top of gate is usually over FRL. The installation requires a spillway structure divided by piers to accommodate the gates. Gate openings are rectangular so streamlining of the gate

shape or its border affects the hydraulic and structural performance.

There are several types of gates used in spillways, for surface flow and for sluices (low or intermediate level outlets). The most common types of gates are: radial (“tainter”), drum, slide, fixed wheel gates and flap gates. Another type are maintenance gates as bulkhead and the stoplogs, these are installed in quiet water (balanced water head) without hydrodynamic forces to close the space and to allow access to service gate for its servicing/inspection/maintenance/repairs.

In India, the common types of spillway's service gates are radial (Figure 3-116) and

vertical lift gates (fixed wheel type or slide type). From the point of view of security, the usual array is a service gate (radial) with a maintenance gate located close upstream to allow access, inspection and maintenance of service gate; however, there are some old installations where maintenance gates (stop log or bulkhead) are not provided u/s of spillway gates.

- Radial gates

The radial gates, as described by USACE, consist “of a cylindrical skin plate reinforced by vertical or horizontal support ribs, horizontal or vertical girders, and the radial arm struts that transfer the hydraulic loads to the gate trunnions. Radial gates rotate about their horizontal axis during opening/ closing operations”. The shape of a radial gate is part of circular (W= width; H= Height), the radius commonly varies from H to 1.5 H.

Operation of gate is carried out by a hydraulic hoist or rope drum hoist (with a steel rope or chain wound on a drum). According to gate position (closed or partially open) acting loads are hydrostatic, hydrodynamic, self-weight, force from hoist and friction at trunnion. The operating systems are manual or electrically operated with different type of hoists as: hydraulic hoists, rope drum hoists (with wire ropes or chains) connected to the gates.

Figure 3-117 shows a typical radial gate and its structure including the traditional hoist . equipment with chain; in relation to this element, it is important to mention that mostly current designs relies on hydraulic hoist. Other safety features are a backup or auxiliary power system (such as an Diesel generator set) to operate spillway gates under unexpected or emergency conditions, and, stoplogs as maintenance gate, as required. As for any mechanical and electrical component, periodic inspection, planned maintenance and test of functionality are important requirements of a gate installation ( See Appendix E). Figure 3-118 shows a stoplogs in closed position upstream of a radial gate.

Figure 3-119 and 3-120 shows Radial Gates in two of India’s very large dam Spillways.

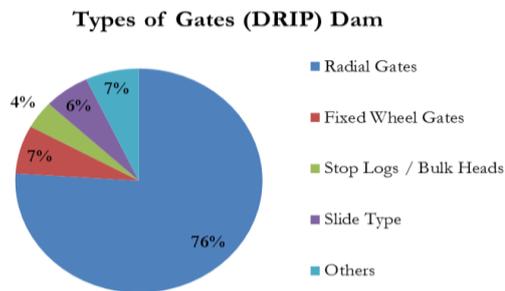


Figure 3-116: Types of gates in spillways of dams within DRIP (India).

- Bulkheads

The bulkhead gates, as described by USACE, are “mechanical features used to isolate the downstream spillway (including regulating gates) from the reservoir or from tailwater, which is done to facilitate maintenance operations and inspection of normally inundated portions of the spillway. The bulkhead is a flat, structurally reinforced gate leaf with rubber seals, which comes in various shapes and sizes to fit a particular control structure. The bulkhead normal-

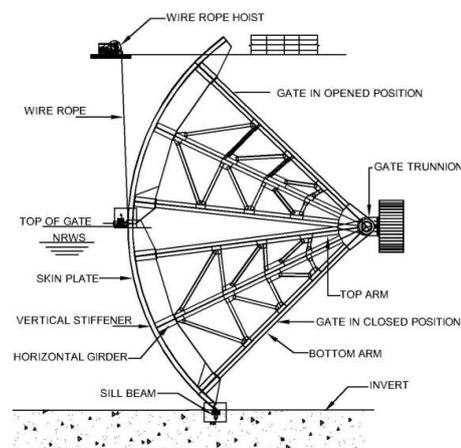


Figure 3-117: Typical Radial (Tainter) gate (USACE)

by fits into vertical gate slots for horizontal flow entry type control structures, such as a gated ogee crest control structure, or it is a top vertical entry type control structure”.

- Stop logs

The stoplog, as described by USACE, is “mechanical feature used to isolate the downstream spillway including regulating gates) from the reservoir in order to facilitate maintenance operations and inspect normally inundated portions of the spillway. Also, stoplogs have been used to temporarily raise a reservoir. Stoplogs consist of individual beams, girders, or multiple beams and plates welded together to make one stoplog. Stoplogs are set one upon the other to form a watertight barrier supported by gate slots for a horizontal flow entry type control structure”.

Figures 3-118 shows a group of metallic stoplogs to isolate a radial gate for repairs; these units are stacked together one over the other to make the closure (USACE).



Figure 3-118: Typical installation of radial gate with stoplog (USACE)

Many spillway gates in India are 50 years old and more and, either need maintenance or periodic operation to verify whether they are in good conditions to operate in the case of a large flood to occur. Present dam design includes provision of bulkhead gates or stop logs for most spillways particularly in the upstream control section element. Further, for some energy dissipaters whenever is feasible, bulkheads or stop logs are also placed at the downstream end of the terminal structure, particularly for the case of stilling basin or any other type of submerged energy dissipater where frequent inspection may require to visualize in the dry the condition of the energy dissipator (Figure B.1-10, Appendix B). Site Specific bulkheads for spillways and deep intakes have been used with reasonable results in a number of facilities (Moreno et al, 2006, Goodwin et al, 2019, Lux, et. alia,2010).

Both Bulkheads and stop logs are sometimes required to be provided in existing spillways. (see Figures 3-119 to 3-122). Feasibility investigations will need to be carried out for the design of the bulkhead or stop log system, lifting equipment type and capacity. However, the feasibility of the same will depend on the modification possible in civil structure viz. spillway piers, crest, bridge, etc.

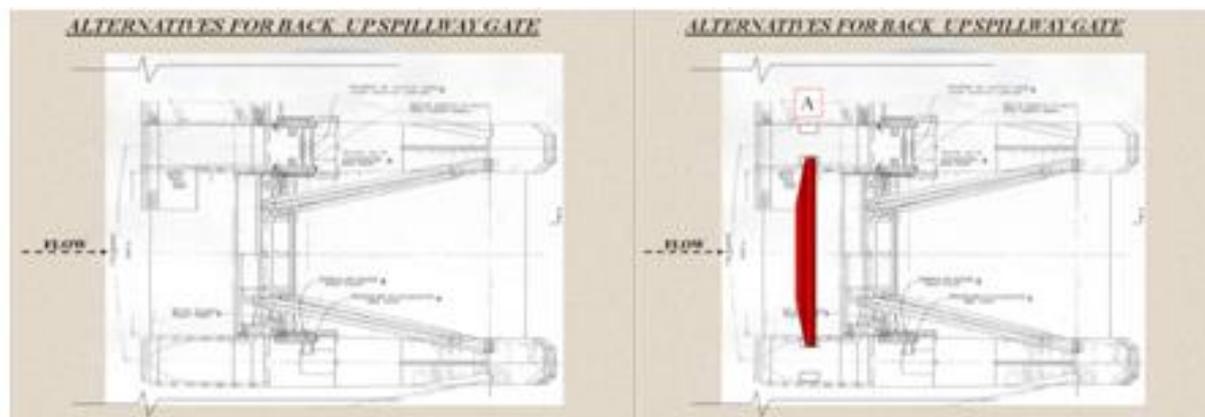


Figure 3-119: Rehabilitation works for isolating spillway gates - Option A-Plan (Stoplog resting on spillway crest)

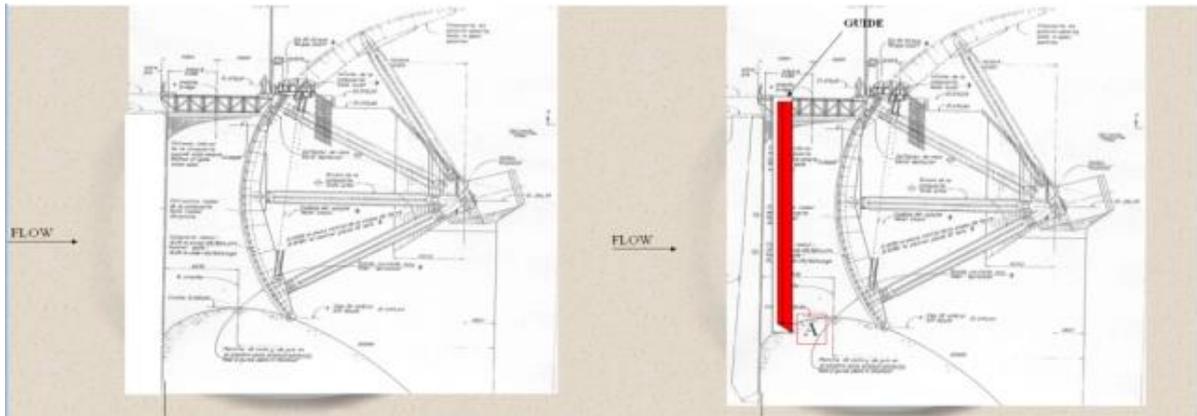


Figure 3-120: Rehabilitation works for isolating Spillway gates - Option A-Elevation (Stoplog resting on spillway crest)

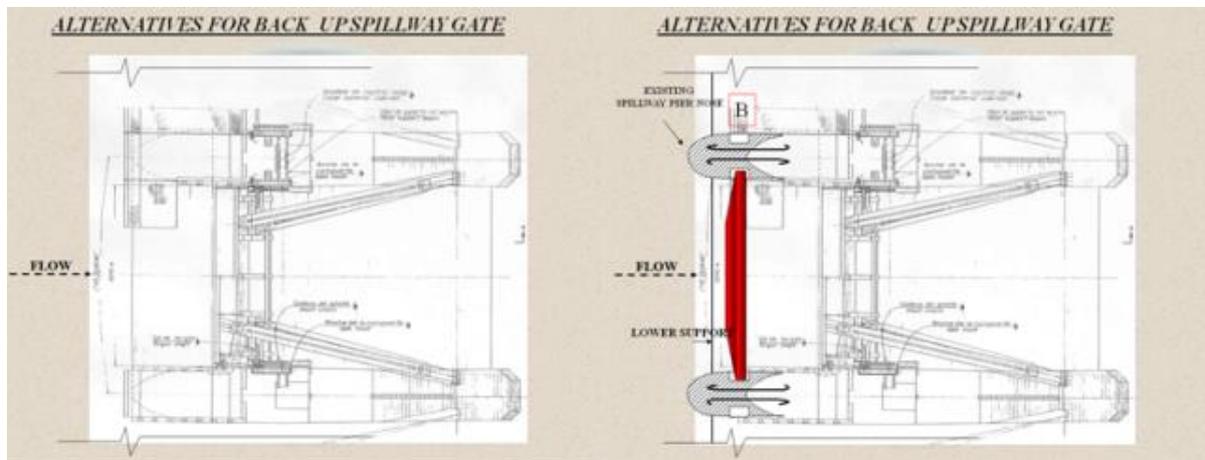


Figure 3-121: Rehabilitation works for isolating Spillway gates -Option B –Plan (Stoplog resting on platform u/s of spillway crest)

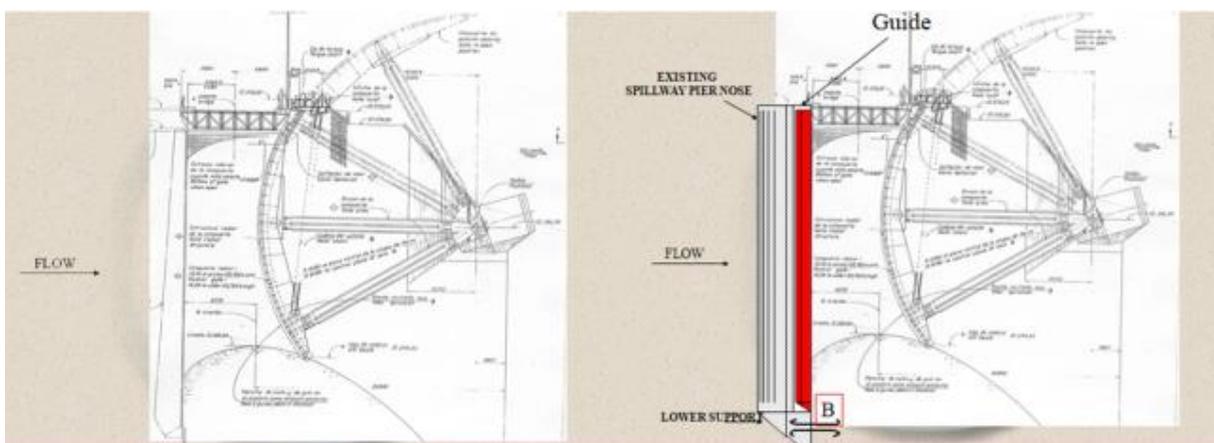


Figure 3-122: Rehabilitation works for isolating Spillway gates - Option B-Elevation (Stoplog resting on platform u/s of spillway crest)



Figure 3-123: Radial gate: hoist equipment by chain and hydraulic (Guri and Caruachi Dams, on Caroni river, EDELCA, Venezuela)

The CWC's manual for "Assessing Structural Safety of Dams" also presents different type of gates and operating equipment generally used in India. Some of the gate installations are shown in Figures 3-123 to 3-127 for reference.



Figure 3-124: Typical installation of bulkhead gate (USACE)



Figure 3-125: Typical metallic stoplogs for a radial gate (USACE)



Figure 3-126: Very large spillway, 27 radial gates, 15.4 m x 14 m, 49,500 m<sup>3</sup>/s, 130 m<sup>3</sup>/s/m each bay  
Gated spillway, Kadana Dam, Gujarat, India



Very large spillway; radial gates, composite dam, 56.4 m high, 18,010 m<sup>3</sup>/s

Figure 3-127: Gated spillway, Ujjani Dam, Maharashtra, India

### 3.3 Hydraulic Safety Assessment of Control and Conveyance Features

For spillways, reliability is related with the ability to accomplish its hydraulic function during its working life; thus, it covers both serviceability and durability, under different situations: usual and unusual. Hydraulic safety is established according to the response of the works to exposure and to actions (or loads), that can cause damages or can affect its performance. In a structure (or any of its elements), the condition is assumed static if it does not change with time; however vulnerability applies to condition of the works “as on today”: new or aged, as designed or modified, well-maintained or poorly maintained. Since this manual refers to existing dams, focus is on old structures or those with certain operative life. For a spillway, hazards come from its exposure to floods (IDF or any other flood) and to flow conditions, the first is the typical hydrological scenario and the second is the response to hydraulic causes.

In a spillway two hydraulic functions have to be complied :

- Capacity to discharge the IDF, without damages to the dam or to any features of the spillway.
- Capacity to convey the discharge to a site downstream of dam without damages to any of its features and to the river environment

Even though the first function is clearly related to the safety of the dam, the second could lead to serious consequences to the owner not only for the cost involved in repairs but also for the potential hazard to the dam if damage extends.

This chapter covers hydraulic safety of control and conveyance features of the spillway, and their response to the occurrence of any of the following loads conditions:

- Due to discharging capacity:
  - Flood greater than original Inflow Design Flood IDF (“as designed”)
  - Inflow Design Flood (IDF)
  - Frequent floods (any discharge lower than IDF)
- Due to flow
  - Any hydraulic action due to passage of any flood in flow regime of high velocity (high energy)

The condition imposed by a flood greater than original IDF in an existing (old) dam is the worst scenario of vulnerability (or the critical failure mode) because of the possibility of major damages to the dam due to potential overtopping and eventual dam break. The spillway requires rehabilitation (upgrading) due to high hazard potential of reservoir, and the risk has to be managed to a defined level by increasing spill’s capacity. This is the main problem to solve for hydrological and hydraulic safety of dam-reservoir system. Potential dam’s overtopping has also been discussed in Chapter 2 of this Manual. On the other hand, even if there is no dam break during this scenario, the spillway is in an overloaded state and any (or all) of its component (s) could be damaged or may even collapse; an uncontrolled discharge would pass downstream causing damages and the benefits from the dam and reservoir may be avenged affected. Annexure “A” of this manual presents Failure’s Modes and its identification, a key activity in the process of dam’s safety assessment.

The dam and reservoir must be able to pass the IDF safely. The total spill capacity (i.e. capacity of all spillways: service and auxiliary, and sometimes, also emergency added together) and the conveyance capacity must be adequate; so, design and rehabilitation should focus on this requirement.

<b>REFER APPENDIX A</b>
To identify probable failure mechanisms as a consequence of inadequate capacity or malfunction of the spillway. See FM-1 to FM-9

For high frequency floods (or even for operative spills from the reservoirs), the lack of available spillway capacity could be a major concern if spillway gates are not in working condition (e.g. gates with structural damages or with functional problems) or if there is some vulnerability in any component that avoids using it during a frequent flood (see Oroville Dam, USA, 2017).

The condition of conveyance of discharge in supercritical flow (high to very high velocity) up to the terminal structure at the toe of the dam or at some site downstream, introduces several scenarios of hydraulic loads either in the control structure or in chute (or tunnel). It may be mentioned that ICOLD classifies as very large those spillways with specific discharge greater than  $130 \text{ m}^3/\text{s}/\text{m}$ .

For these load conditions all modes of malfunctioning or failure are triggered by hydraulic actions; adverse response is expected in a structural element, so rehabilitation's measures are mainly structural. Other aspects related with this hydraulic functioning have to do with frequency, duration and repetition of the loads acting on the elements. This means that incidents not only happen in one large event of flood but also could occur by accumulating effects or damages from many frequent events (much lower than IDF) as "progressive failure" during operational life, then, suddenly, a structural element may fail or break.

In some cases, these failure's modes of spillways due to malfunctioning of its features other than the control structure are defined as "non-critical" because they do not produce incidents that endanger the dam; however, damages can be serious and repairs could be complex and costly. As said before, some modes of failure could become critical for example in case of scour/erosion progressing towards the dam. As a matter of fact, there are examples that became lessons about safety of spillways. It is important to mention that surveillance and maintenance of spillways are fundamental activities for management of the risk due to hydraulic

actions. Surveillance includes the ability to detect deficiencies or hidden damages and the effects of their progress to a future incident.

The rehabilitation of a spillway encompasses three basic activities related to analysis of potential adverse responses:

- Definition of IDF while carrying out flood review studies as per the CWC publications: "Guidelines for Assessing and Managing of Risk Associated to Dams", "Guidelines for Selecting and Accommodating Inflow Design Floods for Dams", "Guidelines for Classifying the Hazard Potential of Dams", IS 11223 Guidelines for fixing Spillway Capacity.
- Study of the spillway design documents, operative programs, geology of the site, incidents during operative life and current physical and functional condition and the state of the spillway, as indicated in CWC publications: "Manual for Rehabilitation of Large Dams", "Guidelines for Safety Inspection of Dams" and "Guidelines for Evaluating Geological Conditions of Dams".
- Analysis of files and field data acquisition to define sensitivity of components of spillway which may be affected, in this case control and conveyance structures. This analysis allows to define effects of hydraulic loads and the adverse responses of elements and credible modes of failures of those elements (or eventual failure of the spillway).

As in new works, rehabilitation follows the approaches of Risk Analysis; the spillway must have adequate capacity to cope with the IDF and should not have fragile elements that affects its hydraulic functioning and stability of its features.

The vulnerabilities of a spillway's components (or elements) can be due to:

- Those components of the spillway with characteristics that limit the capacity of discharging and conveying the water releases.
- The physical condition of various components of the spillway, concrete surfaces and local structural details, and their effects on flow.
- The human's dependent activities. This vulnerability has to do with the "soft" component of any structure such as training of personnel, surveillance, maintenance, operation, control and monitoring. This chapter only mentions these topics, since details are included in the following CWC's publications: "Guidelines for Preparing Operations and Maintenance Manuals for Dams", "Guidelines for monitoring and improving the health of dams", "Manual for Assessing Structural Safety of Dams", "Guidelines for Safety Inspection of Dams" and "Guidelines for Instrumentation of Large Dams".

All possible hydraulic vulnerabilities of spillways are to be examined by assessing the cause and the response of the structural elements of the control structure or conveyance features. The probable modes of failure are presented in Annexure "A" of this Manual. The main vulnerability which is "inadequate spillway capacity" is included but ample hydrological or structural explanations are included in other CWC Manuals.

### **3.3.1 Inadequate capacity of the spillway**

As mentioned, in India an important activity is to upgrade spillway's capacity to manage the risk and improve hydrological and hydraulic safety of existing and old dams.

First of all, the design flood is to be reviewed; then flood routing studies are to be carried out to determine the maximum water elevation (MWL) for the revised flood; then

the adequacy of freeboard available over the revised MWL is checked.

In rehabilitation of spillways, the first action is to re-evaluate its actual capacity, especially in old dams without data. The spillway capacity depends on hydraulic head and physical and hydraulic characteristics of approach channel, inlet and control structure, d/s submergence, reliability of gates in case of controlled spillways, etc.

There are several aspects to be checked in order to establish the strategies to increase its capacity. By reviewing the equations of Discharge's Curves, the discharge can be increased if:

- Weir can function with more head (H)
- Coefficient of discharge is increased
- Length of weir is increased
- Installation of gates is modified: type and number of gates
- Operating rule of gates is modified.

The options related with accepting greater head (increase in MWL) depends on adequacy of freeboard available. On the other hand, the functioning of existing spillways must be checked for greater head and larger discharge - not only the control structure but features downstream i.e. the conveyance, terminal and exit channel. Commonly, accepting greater reservoir water elevation is the first alternative to consider if increase in discharge and head is not so significant, and no other scheme is possible.

Chapter 2 "Dam and Reservoir" presented those options to deal with the increase of reservoir water level, such as: (1) Heightening of the dam (2) Provision of u/s solid parapet wall and (3) Adapting embankment dams for overflow (protected sectors of the dam as non-conventional overflowing weir). This part of manual focuses in dealing with IDF or higher floods (with or without optional measures to manage water elevation in the reservoir) and its effects in the spillway,

to guarantee hydraulic security of the reservoir-dam system.

Other conditions which could be modified to improve the flow pattern over a weir and to increase the coefficient of discharge, could be improvement in the condition of approach channel by excavation/de-silting locally, and in the condition of crest of weir, piers and abutments or by repairing damages, if any. The common problems related to approaching flow are: loss of area due to obstruction and lack of uniformity of flow on the weir. The obstruction can be mainly due to accumulation of floating debris and deposits of material from bank sliding in the approach channel, both conditions are addressed in Chapter 2. The streamlining of flow in front and over the weir (nappe) is affected by geometry of approach channel and inlet, and by irregularities/damages in the flow surfaces. The added effect of these conditions produces separation zones and greater energy losses that modify the discharge's coefficient ("C") and/or the effective length of the weir. In order to optimize the weir's efficacy, flow condition in the inlet and over the weir should guarantee best use of the available physical length. By doing repairs on concrete surfaces or adding guide elements at the inlet, local flow conditions could be improved including improvement in "C"; but usually major adaptations/repairs on spillway surface, piers and abutments are not possible; these contour deficiencies could be present in old dams. Thus, among the mentioned conditions, the reliability of the reservoir system to release floods can be summarized in availability of effective length of weir for the expected discharge.

In controlled weir, vulnerability due to limited discharge capacity is related to number of gates and their functional condition.

It is important to mention that since increasing spill capacity of reservoir is an important matter for dam's safety/security, it is necessary to examine all possible alternatives for the additional spillway before taking a final decision. Usually, for rehabilitation of an

existing spillway, the maximum discharge is defined and the extra required discharge is to be managed with an auxiliary spillway; then capacity of conveyance and other hydraulic loads are evaluated for that "spillway design discharge".

In summary, increasing capacity to upgrade the spillway capacity depends on availability of physical space or available spill's length in the existing site or in some other location (topographical and geological investigations are to be carried out for establishing feasibility of the site), availability of downstream channel for carrying the waters, hydraulic efficiency (passing more discharge with less or little increase in existing MWL) and cost.

### **3.3.2 Responses of conveyance structures to hydraulic actions**

In the conveyance feature (chute or tunnel), vulnerability to hydraulic actions depends on their physical condition, geometry of the section and with surface's irregularity or damages or defects due to concrete degradation along the structure that can affect stability of flow. The adverse response of structures to those different hydraulic "loads" are local (or extended) and progressive since they could create a cycle of damage in various elements such as slab, walls, foundation etc. of the conveyance feature.

These hydraulic loads are present for any discharge, even those smaller than IDF; there are examples of chutes and tunnels with substantial damages due to these dynamic hydraulic loads during frequent discharges. It is important to mention that the research about occurrence of these hydraulic loads in supercritical flow and their effects since 1970-80 onwards, has helped in the understanding of the subject and to evolve an engineering criteria for estimation of hydrodynamic pressures; however, there are still some concerns about this topic, especially, in very large spillways. Dams older than 40 to 50 years are more likely to suffer damages by these hydraulic actions.

1) Vulnerability related to geometry of the channel section (chutes and tunnels)

The geometry of chute's section and its longitudinal profile influence flow conditions, for example, converging and diverging walls and vertical curves along the chute's profile, can generate a particular flow's pattern as shock waves or super elevation, and possibility of separation zones of flow from the invert. There are established criteria for shapes and angles required at these changes of chute's width and curves, but in rehabilitation, modifications are not possible so concerns are about height of walls and local surface degradation in zones close to these changes in geometry.

When increasing the discharge's capacity of the control structure (to updated IDF), an important feature to check along the chute is its height of walls, which should cover bulked water depth with enough security margin. The chute must have enough conveyance's capacity and lateral confinement for the safe passage of IDF, without overflow of walls, especially in its section close and downstream to the control structure where usually there is a reduction of wall's height.

Freeboard of walls has to be checked along the chute according to water surface profile with change in surface roughness, air bulking, shockwaves and splash. In relation to air in the mass of water, it is important to check bulking of flow due to insufflation of total air conveyed as "air entrained" (air being transported by the flow as bubbles) and "entrapped air" (air transported with the flow in the roughness or waves of the water surface) (Wilhelms and Gulliver, 2005). The depth of water with air can even increase more than 2-fold according to flow condition, the slope and roughness of the chute. There are some expressions to calculate freeboard, which include air bulking, waves, uneven distribution of flow and splash, but it is important to know their velocity's range of use and to revise their application to very high velocity flow. In cases, using hydraulic models is a

need, as presented in Annexure "D" of this manual.

If overflow of chute's walls occurs then the backfill material becomes the fragile component since it can be eroded and washed away; if the erosion's process is long enough (according to duration of flood passage) then chute is vulnerable to undermining, channel's structure can fail and damage can progress leading the control structure due to head cutting of foundation material, especially in cases of weak rock or soil strata.

In tunnels, as a confined section with non-pressurized flow, the locations to be evaluated are those where there are changes in directions, in particular, vertical bends where damages can occur as a result of local variations of flow pattern due to centrifugal force. Those locations should be inspected for concrete damages.

Figure 3-128 shows the pattern of high velocity flow in a chute with air bulking and waves, also the typical wave called "rooster tail", downstream of piers of a gated weir. Figure 3-129 shows a case of spillway's collapse due to overflowing of chute by a flood greater than IDF.

2) Vulnerability due to condition of the surface of concrete

The surface of the chutes and tunnels, may have physical irregularities (which change with time) resulting from the displacement of elements, aging and degradation processes of concrete, which depend on the materials used and their resistance, temperature variations, chemical processes, foundation material and actions of the environment to which it is exposed. In spillways, the environment corresponds to turbulent high-speed flow, which is not only especially aggressive but its impacts are cumulative since deterioration processes on surfaces cause the flow's effect to intensify, creating a cycle of progressive damage to the concrete element. The degradation of concrete due to flow encompasses two processes: erosion (abrasion) and effects



Bulking air, waves and rooster tail waves at piers.

Figure 3-128: Flow pattern of high velocity flow in a chute.

due to changes in local pressure (cavitation and pressure changes or stagnation point).

- Abrasion (or erosion due to water + solids)

This type of erosion occurs due to high-speed turbulent flow parallel to the surface, with transport of abrasive sediments (silt, sand); it also includes the impact and wear effect of heavy floating debris and transported or dragged hard fragments (stones, boulders, cobbles, gravel) in contact with the concrete surface (invert or walls). The abrasion's mechanism (friction + rubbing) disintegrates the cementitious paste of concrete, wearing/releasing of the coarse aggregates, the wear (removed material) has a pattern nearly parallel to the surface, extended, continuous and with appearance of ripped material with rough finish. As a reference, the threshold velocity of flow to abrasion process in concrete is around 12 m/s.



Overflowing of chute's walls, erosion of abutment, undermining of chute, head cutting and collapse of spillway (see Case Study in Appendix).

Figure 3-129: Chute overflow. El Guapo Dam, 1999, Venezuela,

The intensity and speed of the process depends on the frequency of passage of water,

suspended sediment and the quality/ durability of the concrete, especially in old dams. In incipient or early stages, abrasion can be controlled with maintenance tasks, but in case of extended flow with time on sections of the chute, the damages can affect the structural element and its reinforcement; so a sound rehabilitation is needed. The resulting uneven surface increases abrasion and facilitates the occurrence of other process, cavitation. Abrasion is more common in spillways where sediment load is large and debris/trash/logs/boulders etc. pass over the spillway. Figure 3-130 shows a scheme of the process. Figure 3-131 shows stages of abrasion on invert of tunnel and chute.

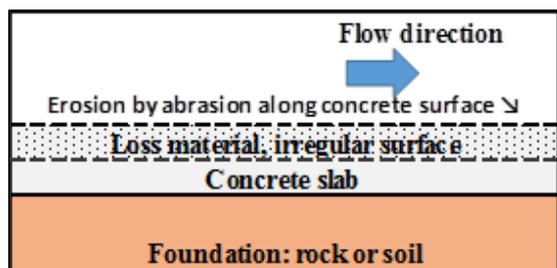


Figure 3-130: Scheme of abrasion process.

There are many examples of spillways with major damages due to abrasion in chute or tunnel. Several cases have been reported in India, especially in the spillways of dams located in the Himalayan region, where the sediment load is heavy and so the damage is intense by abrasion or impact to concrete elements. Some examples are Bhakra, Maneri and Ichari dams (see Figure 3-132).



Loss of concrete element and cover of reinforcement bars are exposed to corrosion.

Figure 3-131: Advanced abrasion process at tunnel and chute inverts.

- Cavitation

This type of erosion or damage occurs when flow pattern changes due to an irregularity in contour surface thus velocity is locally increased and consequently pressure drops; if vapor pressure of water is reached, there is bubble (cavities or voids) formation that

travel in the water and rapidly collapse in a higher pressure zone downstream. If collapse occurs on concrete contact, high pressure shock waves that propagate at speed of sound impact the surface generating erosion as pitting. The repeated impacts wear the concrete surface in an irregular fashion, as pits becomes holes then the process intensifies. Cavitation can occur with the passage of smaller floods than IDF.

Typical location along the chute to start a cavitation process is an offset at the surface as a concrete slab displacement in a contraction joint, a protrusion of joint's filler, an irregular finishing of the surface (abrupt roughness), angular (non-gradual) changes in surface, cracks or other damages on concrete surface. As mentioned previously, abrasion damages can also cause cavitation. Once the process of cavitation is initiated, the time required up to failure of structural element, will depend on duration and repetition of flow, and characteristics of concrete. In extreme cases, if concrete slab fails then foundation becomes the fragile element due to its exposure to erosion, so intensity of the damage will depend on erodibility of the base material. Figure 3-135 shows a scheme of process of cavitation due to an offset of concrete surface.

REFER APPENDIX B.4
Case Study – Maneri Dam Repair of Spillway and its Energy dissipater severely damaged by rolling boulders during floods.

In order to establish the cavitation's potential in any location along the chute, an index ( $\sigma$ ) is calculated by using local properties of flow. For the general case of a curve surface of chute:

$$\sigma = (y \cos \theta \pm y \frac{v^2}{gRc} + P_b - P_v) / (\frac{v^2}{2g})$$

Which for flat surface reduces to:

$$\sigma = \frac{P_o - P_v}{\rho \cdot \frac{v^2}{2g}}$$

Where:

$\sigma$  = Cavitation index (dimensionless)

$P_o$  = Absolute pressure at concrete surface ( $N/m^2$ ) = local barometric pressure + hydraulic gauge pressure.

$P_b$  = Local barometric pressure ( $N/m^2$ )

$P_v$  = Vapor pressure ( $N/m^2$ ) (varies with temperature)

$y$  = Depth of flow (m)

$\theta$  = Angle of chute with horizontal ( $^\circ$ )

$R_c$  = Radius of curvature of chute (m). Concave surface (+) or convex surface (-).

$\rho$  = Density of water ( $N/m^3$ ) (varies with temperature)

$V$  = Average flow velocity of flow at the section (m/s)

Table 3-14 presents the velocities of flow and its corresponding cavitation index  $\sigma$  (temperature of reference 20  $^\circ$ C), used to propose a classification of risk of damages by level of cavitation.

For initial estimation, the cavitation index becomes critical (threshold to cavitation damage) in the range 0.2 - 0.5. Singular roughness (or asperities), local (abrupt) or distributed, as: offsets, irregular concrete surface or large features, are sources that cause separation of flow from boundary and favor occurrence of cavitation damage for larger values of cavitation index than those in Table 3-14, even greater than 1. Figure 3-134 shows typical asperities on concrete surface. As basic initial evaluation, sections of chutes with high velocity ( $V \geq 15$  m/s) are expected to suffer most damage; also, roughness of concrete surface, with abrupt offset ( $>5$  mm), could be the trigger cause of cavitation damage in station of high velocity of flow. As an example, Figure 3-136 presents values of  $\sigma$  for irregularity "Type 1a" after Liu.

For rehabilitation of a chute, after asperities survey and closed surface inspection, sec-

tions with damages or potential damages are



Concrete Gravity dam, 226 m high. Spillway Stilling basin was damaged (see case study in "Manual on Rehabilitation of Large Dams")

Figure 3-132: Bhakra-Nangal Dam (Himachal Pradesh).

defined. A hydraulic analysis of water profile is done for selected discharges up to updated IDF. Cavitation indexes are used to estimate the potential risk of cavitation, by comparing them with typical indexes of each type of irregularity that induces cavitation

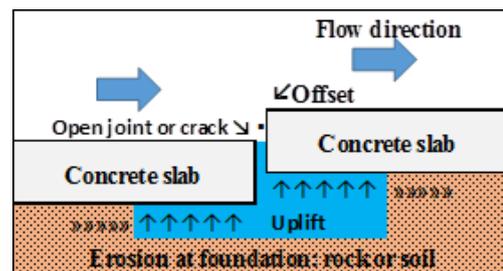


Figure 3-133: Scheme of stagnation point creation and displacement of concrete slab

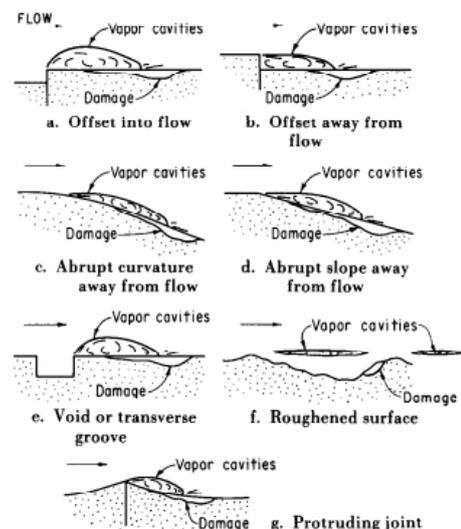


Figure 3-134: Typical irregularities on surface

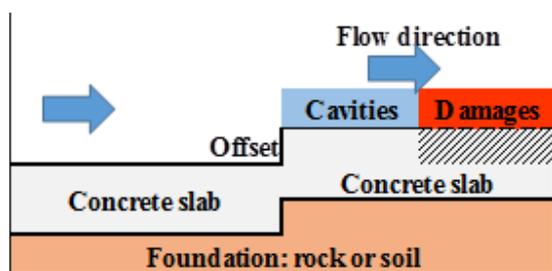


Figure 3-135: Scheme of cavitation process.

Cavitation can be controlled with: (1) Adequate geometry of chute, (2) Uniform and smooth concrete surfaces (within tolerances) (3) Using high resistant concrete special and other products, and (4) By adding air (in low concentration) to water. In rehabilitation of existing spillways, options (2), (3) and (4) are commonly used. Air could be added naturally by turbulence of flow, but for cavitation index lower than 0.25, air entrainment must be forced by constructing an aeration ramp or slot at the section of the chute where pressure drops or where it is expected to occur. Efficacy of using high resistant material will limit the process but depending on duration and frequency of exposure to floods.

Some cases of cavitation damages in spillways (tunnel and chute) are Glen Canyon Dam (USA) and Guri Dam (Venezuela). Figure 3-140 shows an example of serious cavitation damages (Level 4) on invert of chute. Figure 3-139 shows the case of major cavitation at tunnel invert, where erosion of rock and collapse of tunnel occurred after

failure of concrete lining.

Level	Cavitation Damage Risk	Range of Velocity (V) (m/s)	Range of Cavitation Index ( $\sigma$ )
1	No Cavitation Damage	$V \leq 5$	$\sigma > 1$
2	Possible Cavitation Damage	$5 < V \leq 16$	$0.45 < \sigma \leq 1.0$
3	Cavitation Damage	$16 < V \leq 25$	$0.25 < \sigma \leq 0.45$
4	Serious Damage	$25 < V \leq 40$	$0.17 < \sigma \leq 0.25$
5	Major Damage	$V \geq 40$	$\sigma \leq 0.17$

Table 3-14: Cavitation damage level (Fadaei Kermani, E. et alia, 2013)

- Hydraulic failure due to stagnation pressure

This type of damage can occur due to two processes, both related to high velocity flow and dynamic pressure fluctuations, in a section of the chute where there is an open (non-sealed) joint, a crack and/or an offset in the concrete slab. In case (1) flow enters through the open joint and produce uplift pressure that moves portions of the slab, and in case (2) flow enter to the foundation through cracks or open joints, which results in internal erosion of the foundation's material and loss of support of portions of the chute.

As soon as slab is moved and a vertical offset opposes the flow, local changes in magnitude and direction of velocity tends to create a stagnation point with increase local pressure and uplift. This process is also called

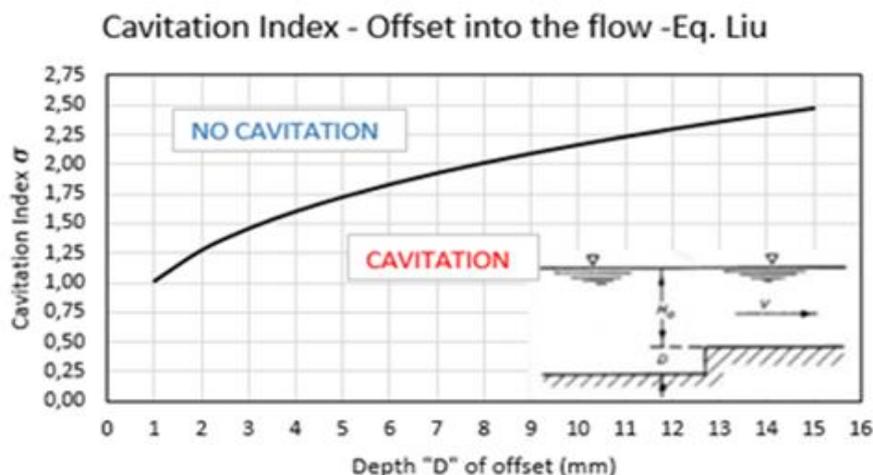


Figure 3-136: Cavitation index for irregularity Type 1 a.

“hydraulic jacking” on the chute. The slab becomes fragile to this load if its resistance (weight of concrete’s slab, reinforcement, anchors to the rock and underground drainage system) is exceeded by the uplift’s load induced by the process. On the other hand, water flowing through the opening follows a path along the contact of concrete-foundation and within the material, resulting in erosion and piping, so in this case there is a combined effect to increase damage up to slab collapse. Erosion will vary according to erodibility of foundation’s material and the existence of a drainage’s system under the slab with an adequate filter’s protection, but, usually, the amount of water entering in the space under the slab exceeds the capacity of drains, so they lose their function of protection. This process of stagnation pressure plus erosion will depend on duration and repetition of floods, in some cases it can develop rapidly, in other cases it could be present but without obvious damage or signs on slab surface. In some cases, there is a non-detected significant damage by undermining and suddenly, in a low flood, the slab collapses. In advanced stages, on erodible material, the process evolves to gully formation with head cutting.

Typical causes of a stagnation point are open joint (without filler or seal), joints with damaged filler or seal, joints with vertical and horizontal offset, and irregularities due to previous processes of concrete deterioration on surface, such as: cracks, damages by chemical reactions in the concrete, damages by freeze-thawing, abrasion, delamination, corrosion of steel bars and swelling or burst of concrete, others. As said before, underground drainage system helps to control the process but high pressure of water seeping down the slab can move filter material, erode foundation material and clog the drains, increasing the uplift on the slab.

Figure 3-133 shows a scheme of hydraulic jacking due to an offset and a joint in the concrete surface. Figure 3-137 presents results of research about flow through joint’s gap with different sizes of offset, according

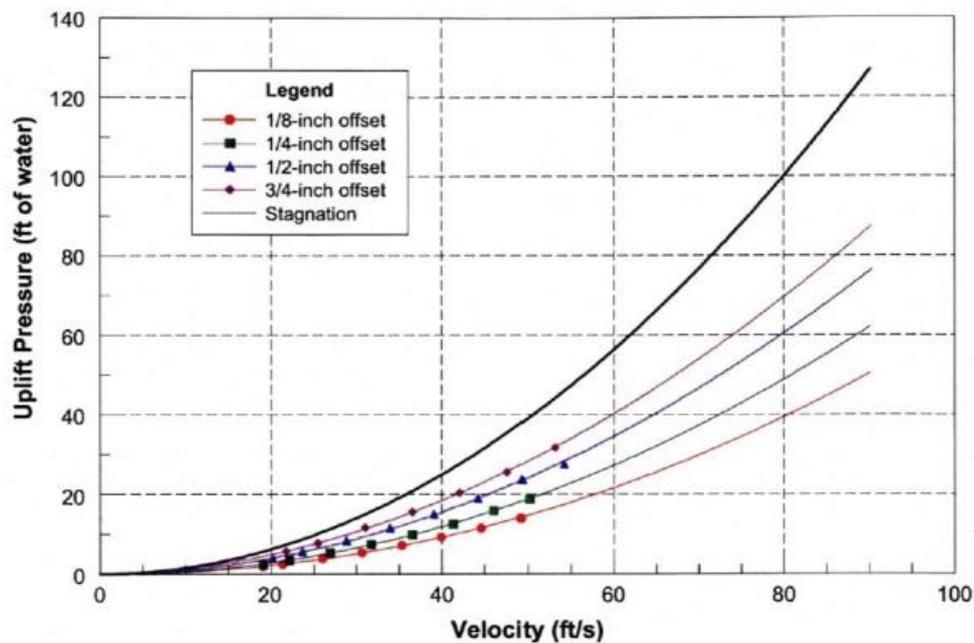
to velocity of flow in supercritical regime; in this graph it is important to notice that tests cover velocities up to 55 feet/s (17 m/s), so caution is recommended for higher velocity as in large spillways (25 m/s and over).

In old dams, there are several conditions related to design criteria, technology and construction that can trigger hydraulic jacking in chutes, such as: use of open joints, properties of concrete, low amount of reinforcement, aging of concrete, deteriorate surfaces, others.

During the technical inspection of the chute, several important surface signs for potential hydraulic jacking’s problem to be checked, are: amount of water flowing from the outlets of drainage system, color of water, solids in water, displacement in joints, cracks in concrete close to borders of slabs, inclination of surfaces, sealing of joints (material and physical condition), others. There are advanced geophysical technology (GPR and others) to investigate foundation of chute for existence of cavities, voids under the concrete slab and erosion process (piping), and others to define physical condition of concrete and reinforcement.

Figures 3-141 and 3-143 present different aspects of slab failure and vertical displacement.

A recent example of chute’s failure with temporal loss of spillway function is the incident in Oroville Dam (California, USA), as a part of destructive process, the induced uplift forces beneath the slab exceeded the uplift resisting capacity and structural strength of the slab. Figures 3-138 on Oroville Dam shows some aspects of this failure, in this case, after the break of concrete slab in a middle steep section of the spillway, there was an intense erosion of medium-quality rock’s foundation. Service spillway temporarily lost its functional ability. The analysis of this incident by an Independent Forensic Team concludes: *“There was no single root cause of the Oroville Dam spillway incident, nor was there a simple chain of events*



Sharp-edged geometry, vented cavity, 1/8-inch gap (3,2 mm) for different offset (USBR, 2007)

Figure 3-137: Mean uplift pressure.

that led to the failure of the service spillway chute slab”.

- Erosion of foundation of chute

In a spillway located out of the dam (in an abutment or any site along the reservoir rim) water seepage from the reservoir through the foundation material is a common issue considered at design. In order, to manage the effects of underground seepage on the chute’s structure, several types of elements are incorporated as: a drainage system under the slab, filter covering the drains and cut-offs. To avoid entrance of water from the chute, contraction joints at the slab are sealed (water-stop), thus to keep foundation isolated. The structural design of the slab commonly considers weight of concrete, keys at joints and anchors to foundation to cope with uplift.

In some cases, especially in old dams, joints are not sealed. In other cases, material used

as seal have lost its function due to aging, temperature changes, fatigue or other causes, so joints are open in many places along the slab. These open joints and also cracks in concrete are passages of water to foundation that can initiate a process, similar to hydraulic jacking, but if the erodibility of foundation material is high as in some soils and weathered or decomposed soft rocks, erosion paths can be established. Water looks for paths to leak under the slab such as contact between concrete and foundation material, cracks and drains, the slab resist the initial increase in uplift so physical signs do not appear on surface; as soon as piping and internal surface erosion progress, spaces under the slab become cavities, water moves freely as a sub- surface flow until slab loses support and fails. Actually, this is a geotechnical mode of failure but due to hydraulic actions, the prevalent process is erosion and undermining below the slab. Figure 3-142 shows a scheme of erosion in its initial stage. A Figure 3-144 shows an example of undermining of chute’s due to this process.



Figure 3-138: Failure of service spillway due to combine destructive processes. Oroville Dam, USA.



Figure 3-139: Cavitation damages plus erosion of rock and collapse of tunnel. Glen Canyon Dam, USA (USBR, ICOLD).



Figure 3-140: Cavitation damages, Level 4. Guri dam, Venezuela (EDELCA).



Figure 3-141: Stagnation pressure. Collapse of slab and offset in a transverse contraction joint with damages (Trojanowski, 2005)

### 3.3.3 Gates

In controlled spillways, gates are key features for hydraulic safety as they help in considerably increasing the spillway discharging capacity. However, their presence in the spillway also adds to the risk to the dam. The main concern around gates is their reliability of operation when needed or in emergencies, since there are many examples of malfunctioning with hazard to the dam and even overtopping due to lack of spillway's capacity. One of the most common rehabilitation work carried out within DRIP relates to repair and updating of gates in spillways.

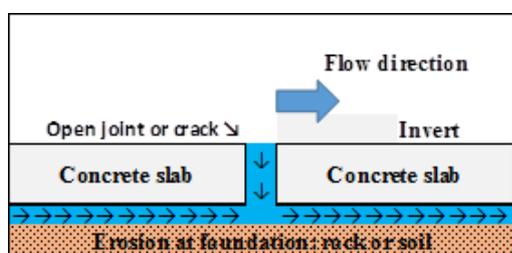


Figure 3-142: Scheme of erosion at initial stage



Figure 3-143: Vertical displacement of concrete slab in a chute due to hydraulic jacking (USBR)

As a hydraulic element, a gate is subjected to hydrostatic and dynamic forces, both requires adequate robustness of the structure and of its hoist equipment. An environment of high velocity flow induces vibration and

repeated load which could be a cause of fatigue and loss of strength of structural elements. Also, gates are subjected to corrosion, friction, debris accumulation and ice effect. Effects of floating debris and ice are discussed in Chapter 2.

The reasons that influence the reliability of gates are:

- Many elements (structural, mechanical or electrical) that can become fragile by aging, corrosion, temperature, impacts, friction and ice.
- Key elements and long term behavior of trunnion, hoist chain or ropes, motor, etc.
- Maintenance dependent
- Need for periodic test of opening and functioning
- Possible obstruction by floating debris
- Possible ice blocking
- Human error while operation
- Interruption of electricity supply

Hydraulic safety depends on a reliable operation of the gates and this, in turn, depends on performance and reliability of its structural, mechanical and electrical components.



Figure 3-144: Failure of a chute's slab due to erosion. Hyrum Dam, USA (USBR, Trojanowski, 2005)

See APPENDIX E  
Operational Safety of  
Hydromechanical Equipment

Refer Appendix E which outlines failures due to gate operation on account of various structural, mechanical and electrical reasons and recommends minimum requirements to ensure operational reliability of gates and hoists. Further CWC's Manual on Assessing Structural Safety of existing dams may also be referred for vulnerability associated with Hydro-Mechanical works (Gates and Hoists).

### **3.4 Rehabilitation Measures for Control and Conveyance Features**

This Manual deals with Hydraulic Safety of existing dams, especially old dams that have to be rehabilitated. In India, more than 80% of 5254 registered dams are more than 50 years old; over 60% of the failures have occurred in the first 10 years of operation, overtopping of earth dams being the most frequent cause due to floods. Within DRIP dams, 70% are more than 30 years old.

This paragraph attempts to cover the measures for upgrading the control and the conveyance features of spillways, in two groups:

- Measures to increase the spillway capacity of the reservoir
- Measures to improve hydraulic performance of conveyance structures (chute and tunnel)

Another CWC's Manual "Assessing Structural Safety of Existing Dam (MASSED)", presents a general list of typical actions to cope with routing of larger floods through the existing dam-reservoir system, and gives the following typical rehabilitation measures to safeguard the dam (structural and non-structural approaches). Also Chapter 2 of this manual presents measures related to the reservoir's safety, some of them included in the MASSED's list:

- Raising the height of a dam in view of higher maximum reservoir level.

- Constructing one or more additional (auxiliary) spillways, fuse plug/breaching sections, flush bars etc.
- Provision of solid parapet wall on the upstream at dam top (where not available) to provide for the revised freeboard requirements.
- Strengthening the crest and downstream face of the embankment to allow some overtopping.
- Collecting more and better data to give advanced warning of adverse conditions and to monitor the response of the dam and reservoir.
- Lowering of the reservoir operating level to increase the flood storage volume.
- Modifying catchment flood characteristics by building flood detention devices or even an upstream dam.
- Increasing dam stability to accommodate higher flood water levels with cable anchors and mass gravity structures.

The other CWC's publication "Guidelines for Rehabilitation of Large Dams" covers measures for repairing the structural components of the spillway.

This Manual deals with the hydraulic options related to increasing spill capacity of reservoirs according to the assigned risk level for hydrological security and measures to improve performance of control and conveyance features.

#### **3.4.1 Measures to maintain hydraulic safety of spillways including measures to increase spillway capacity**

Any blockages and damages in spillway structure and control gates are required to be removed/repared to retain the existing spillway capacity and to avoid further incidents.

Blockages in spillways have been discussed in Chapter 2 of this Manual.

Rehabilitation measures, including the causes for the same and the materials to be used, are dealt in the “Manual on Rehabilitation of Existing Dams” and “Manual for Assessing Structural Safety of Existing Dams”.

The vulnerability of the dam to failure due to overtopping on account of increase in design flood can be reduced by the following two structural approaches:

- Add another spillway as auxiliary or emergency spillway.
- Upgrade the existing spillway.

The first approach is possible if there are appropriate sites (topography, geology, environment and cost) for one or two new spillways. This solution is a completely new study and design according to Indian (BIS) and International Standards, out of the scope of this manual.

For upgrading of existing spillway there are two options:

- Increasing the length of weir is a good option if there is space to do it or if type of existing weir can be modified. Also, broad crest weirs can be modified to ogee crest weirs, if un-gated.
- For cases in which limited length is available, there are options of a different kind that can be adapted to available space; these were formerly presented as non-conventional and recent weirs (generally applicable if existing spillway is an uncontrolled spillway).

Table 3-15 shows rehabilitation measures for improving the hydraulic safety of existing spillways including measures for increasing the spillway capacity.

### **3.4.2 Comparison of options to increase spillway capacity**

Comparison and selection shall be based on hydraulic performance and it shall also be a

cost- effective selection. The design shall be based on the latest or updated design flood (IDF). Various factors to be considered are:

- Topography: Existence of suitable sites (abutments or saddles in reservoir rim) for a new spillway
- Geology: Rock conditions at those sites for foundation of new structure
- Environment: Capacity of water courses for reservoir’s releases.
- Type of dam: Embankment or Concrete, possibility of increasing crest level (dam top).
- Existing (service) spillway: Possibility to enlarge its capacity
- Construction: Available space, access to site, risk.
- Cost

A detailed analysis of the options (advantages and disadvantages), allows to screen and to reduce to those to be further evaluated. During the hydraulic study, an important stage, is to work out discharge’s curves to compare the discharge that can be passed at a reservoir level for each option, which could be a reason to eliminate some of them. Finally, the cost- effective solution results from a combined detailed hydraulic analysis and economic comparison. Figure 3-145 (from USBR) shows an example of a first stage hydraulic study to define and to compare capacities of six new auxiliary spillways along with a dam’s height raise, as a part of dam safety improvement.

### **3.4.3 Measures to improve functioning of conveyance feature from control to terminal structures**

Measures to ensure satisfactory functioning of chute or tunnels are required to be taken on a periodical basis in order to guarantee the hydraulic behavior, integrity and stability of the conveyance structure. They are grouped as per damages caused due to dif-

ferent types of hydraulic actions. Almost all measures to rehabilitate chutes and tunnels are structural, but loads to be considered will be based on hydraulic loading conditions, taking into account the complementary ele-

ments like under drains, treatment at joints, concrete surface, etc.(see Table 3-16).

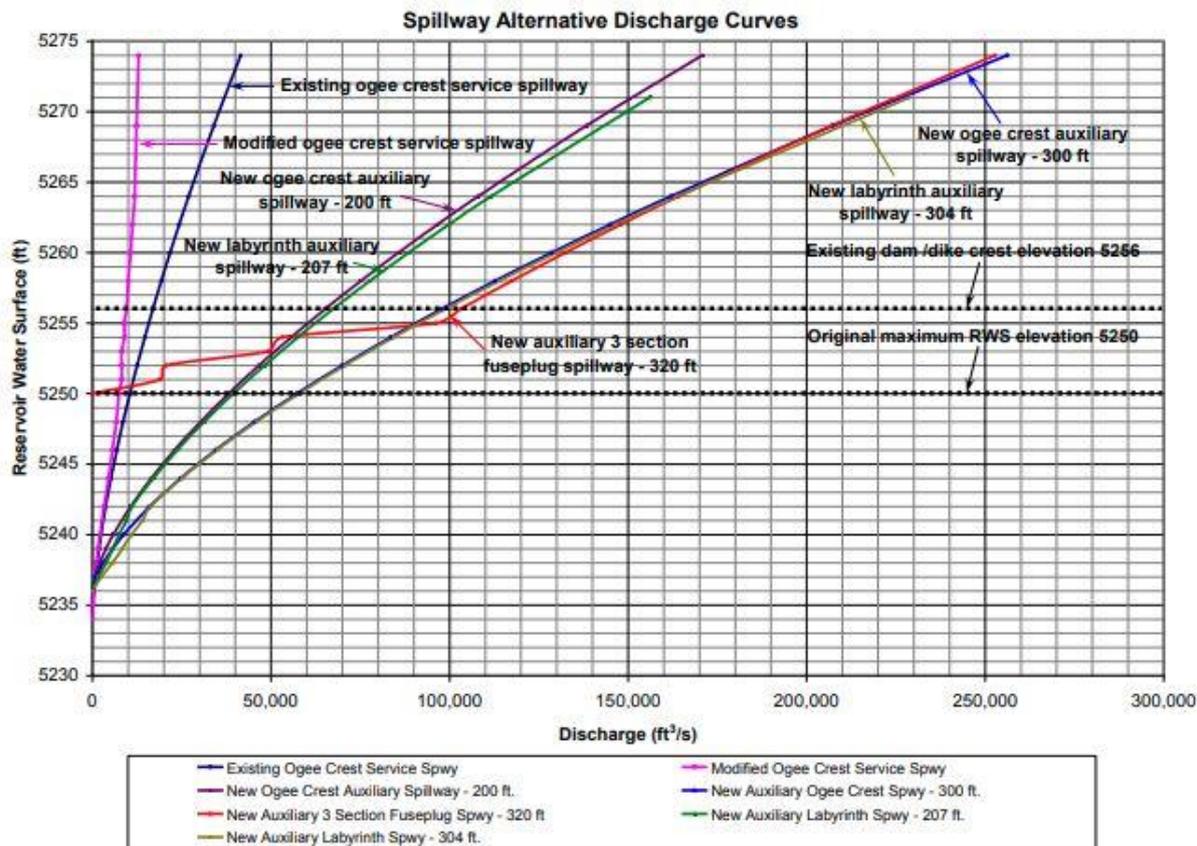


Figure 3-145: Spillway alternative discharge curves. (Example from USBR).

Hydraulic Action	Adverse response of hydraulic or structural element	Rehabilitation measure
Obstruction in approach channel by floating debris	Loss of hydraulic area and increase of energy losses in approach channel	As given in Chapter 2 "Dam and Reservoir".
Obstruction of inlet work by floating debris, around piers and radial gates	Loss of hydraulic area at the inlet. Potential damages and clogging of gates and hoist equipment.	As given in Chapter 2 "Dam and Reservoir".
Ice blocking against radial gates	Loss of hydraulic area at the inlet. Potential damages, obstruction and limited movement of gates. Difficulty to operate hoist equipment.	As given in Chapter 2 "Dam and Reservoir".
Obstruction of approach channel by slipped material from banks	Loss of hydraulic area and increase of energy losses in approach channel	Major maintenance activity or Geotechnical stabilization of bank or slope. As given in Chapter 2 "Dam and Reservoir".
Non uniformity of flow in approach channel and	Reduction of effective length of spillway. Vortex formation.	Incorporate guide walls and training works to streamline flow towards weir, if possi-

Hydraulic Action	Adverse response of hydraulic or structural element	Rehabilitation measure
inlet		ble.
Deterioration of concrete surface at inlet and control structure	Separation zones and turbulence. Reduction of effective length of spillway	Repair concrete surfaces with appropriate materials, improve surface finishing. See “Manual of Rehabilitation of Existing Dams”.
Lack of reliability of gates due to structural, mechanical or electrical issues	Operative vulnerability. Lack of spill capacity. High potential hazard for the dam and spillway	Upgrade/Repair of gates. See Manual “Assessing Structural Safety of Existing Dams” and “Manual of Rehabilitation of Large Dams”
Higher floods. Limited capacity of spillway.	Operative vulnerability. Significant hazard for the dam and/or spillway	<p>Structural Measures:</p> <ul style="list-style-type: none"> <li>• Provision of solid u/s parapet wall.</li> <li>• Increasing height of dam</li> <li>• Provision of Additional Spillway                             <ul style="list-style-type: none"> <li>- Conventional Spillways (Gated /Ungated), Flush bar, etc.</li> <li>Unconventional Spillways as discussed in this chapter of the Manual.(Labyrinth spillway, Piano Key Weirs, Fuse Plug, Fuse Gates etc.)</li> </ul> </li> <li>• Overflow and downstream protection to dams to allow some overtopping. Stepped spillway on embankment dams. (Discussed in Chapter 2 and in this chapter of the Manual).</li> </ul> <p>Non-Structural Measures:</p> <ul style="list-style-type: none"> <li>• Flood Forecasting with sufficient advanced warning.</li> <li>• Lowering of reservoir level to increase flood storage.</li> </ul>

Table 3-15: Rehabilitation measures for improving the hydraulic safety of spillways including measures for increasing the spillway capacity.

Hydraulic Action	Adverse response of hydraulic or structural element	Rehabilitation measure
More discharge to be conveyed (for any option of upgrading the control structure)	Overtopping of chute’s walls, Erosion of backfill, Instability of wall, Erosion and undermining of wall and slab - resulting in failure of chute, eventual head cutting.	For the new water surface profile, increase chute’s wall height as needed. (See Manual “Assessing Structural Safety of Existing Dams” and “Manual of Rehabilitation of Large Dams”)
Changes in geometry of the section of the chute (Horizontal transitions and vertical curves)	Local effects due to high velocity flow in changes of width and vertical curves, damages in local concrete joints.	See Abrasion and cavitation
Abrasion	Deterioration or damage to concrete flow surfaces, corrosion and/or loss of reinforcement, increase in roughness, irregularity of section of chute, potential source to trigger cavitation.	Concrete surface restoration or covering with appropriate material and technology. Structural restoration of sections of chute’s slab. (See Manual “Assessing Structural Safety of Existing Dams” and “Manual of Rehabilitation of Large Dams”)
Cavitation Low pressure section along the chutes and	Damage to concrete, surface of slab and reinforcement, Potential slab break, in extreme cases movement	Concrete surface restoration, finishing and protection with appropriate material and technology. Concrete

Hydraulic Action	Adverse response of hydraulic or structural element	Rehabilitation measure
surface irregularities that disturb flow	of fragments or collapse of slab, potential erosion of foundation material.	joints repair. Demolition and structural restoration of sector of slab, in extreme cases. Check for air in the water. Add aerator in sections of chute as needed. Restoration of erosion gully under the slab. (See Manual “Assessing Structural Safety of Existing Dams” and “Manual of Rehabilitation of Large Dams”)
Stagnation pressure - Hydraulic jacking High pressure fluctuations along the chutes	Uplift and displacement of slab, foundation erosion, damages to underdrain system, piping, collapse of slab, gully formation.	Concrete surface restoration with appropriate material and technology. Concrete joints repair (seals). Demolition and structural restoration of sector of slab, in extreme cases. Restoration of erosion gully under the slab. Restoration of drain system (See Manual “Assessing Structural Safety of Existing Dams” and “Manual of Rehabilitation of Large Dams”)
Foundation erosion Open joints or cracks	Scour of slab, piping, undermining and collapse of slab, damages to underdrain system, foundation erosion, gully formation.	Concrete surface restoration with appropriate material and technology. Concrete joints repair (seals). Demolition and structural restoration of sector of slab, in extreme cases. Restoration of erosion gully under the slab. Restoration on foundation material with selected and compacted soil or massive concrete. Restoration of drain system (See Manual “Assessing Structural Safety of Existing Dams” and “Manual of Rehabilitation of Large Dams”)

Table 3-16: Rehabilitation measures for improving hydraulic safety of conveyance structure

### 3.5 Lessons

In this chapter several examples and cases related with hydraulic safety of spillways have been used to support explanation of processes and probable modes of failure. Numerous incidents in spillway have affected operation of the reservoirs, have compromised the integrity of the dams or produced dam’s overtopping and failure. Below is a succinct account of four emblematic cases where an incident in spillway generated serious damages in reservoir-dam or failure of the dam. These cases are: Tous (Spain),

Oroville (USA), Folsom (USA) and Machhu II (India). Information regarding the above was taken from Association of State Dams Safety Officials (ASDSO). Also, there three other case studies in Appendix B.3 of this Manual: Guri (Venezuela), El Guapo (Venezuela) and Taum Sauk (USA).

**REFER APPENDIX B**

Where 3 examples of failures in the spillway and chutes are discussed: Guri Dam ; El Guapo Dam and Taum Sauk Dam.

• **Tous Dam (Spain, 1982)**

Location: Province of Valencia, Spain, on Jucar River. Dam: Composite Dam, 70 m high Dates: Built in 1959-1978, Failure in 1982 Spillway: IDF (design) 7,080 m<sup>3</sup>/s (500 years flood) controlled by three radial gates, each of size 15.2 m x 10.7 m. (Figure: 3-146)

Incident: Intense storm and heavy rain event delivered a depth equivalent of the average total annual rainfall, within a 24-hour period. Inflow peak estimated up to 9.920 m<sup>3</sup>/s. Reservoir level rose approximately 1 meter over the embankment dam crest. Hydrologi-

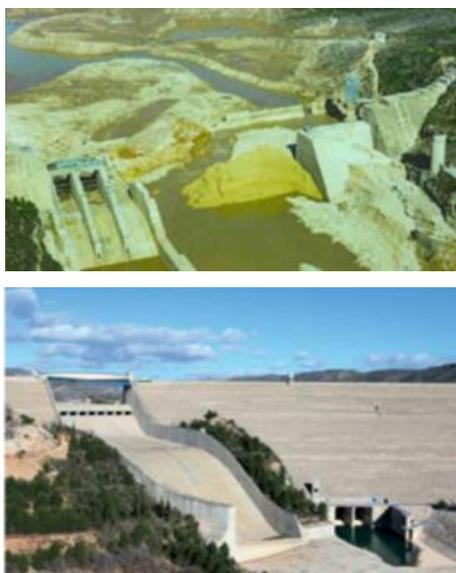


Figure 3-146: Tous Dam. Service spillway failure anew dam. (Dragados, 2015)

cal and operative causes contributed to failure of the dam by overtopping. Power to the electrical grid was lost in the early stages of the storm, thereby rendering the spillway gates inoperable; the only operable power generator available at the site was underwater before operator arrived, other access to emergency generator was flooded. Alert communications failed. Overtopping and dam break occurred in about 12 hours. The reservoir level dropped about 18 meters in 4.5 hours. Dam failure led to a peak breach flow of 15,590 m<sup>3</sup>/s which was a devastating flood with economic and environmental damage, evacuation of roughly 100,000 people, indirect consequential im-

pacts to roughly 300,000 people, and 8 lost lives directly attributed to the dam failure.

Lesson: Significant changes in dam safety standards, emergency communications and risk management; Improvements in flood management policy and strategies in Spain and throughout the European Union; Development and adoption of the early stages of risk-based dam design, flood management policies, development of systems to improve understanding of flood risks, development of emergency action plans, and redundant/reliable communications systems for both flood and all-hazards emergency situations.

The dam was rebuilt in 1996 in the same location, but with significant modifications to address previous deficiencies. For flood management and spillway design the following criteria were used: (1) Increase in dam height to 110 m, providing a reservoir capacity increase of 4.7 times for additional flood capacity, and (2) Ungated spillway designed to safely route the full Probable Maximum Flood (20,000 m<sup>3</sup>/s).

• **Oroville (USA, 2017)**

Location: California, United States of America, on Feather River.

Dam: Embankment Dam, 235 m high

Dates: Built in 1968, Incident in 2017

Service Spillway: Maximum historical recorded flood 4,535 m<sup>3</sup>/s. Controlled by eight radial gates and a chute 915 m long.

Emergency Spillway: It has two sections at its crest: a concrete ogee weir (15 m high) and a broad-crested glacis; discharge is directed down the hillside over natural terrain to the river below.

Incident: In the service spillway when the spill was about 1,490 m<sup>3</sup>/s, the chute suddenly experienced failure and removal of a section of the concrete slab about halfway down the chute. This was immediately followed by rapid erosion of the foundation

and adjacent ground, and progressive failure and removal of the chute slab in the upstream and downstream directions. In an effort to monitor and control the damage to the chute while managing the reservoir level, adjustments were made to the chute flow, but major storms in the large watershed ultimately resulted in the reservoir rising until the crest of the emergency spillway was overtopped for the first time in its history, four days after the chute damage initiated. Maximum flow from the emergency spillway was about 355 m<sup>3</sup>/s (less than 4% of its design capacity), operators were shocked to see that the hillside was eroding along with rapid headcutting due to the overflow. So, there was a risk that the emergency spillway could fail due to undermining thus resulting in downstream flooding due to uncontrolled release of the reservoir. This risk prompted opening of the gates for the service spillway in order to increase the chute flow to about 2,835 m<sup>3</sup>/s and lower the reservoir level. The conveyance and terminal structures of the service spillway were lost. Evacuation of 188,000 people.

Lesson: Better understanding of a mode of failure of a spillway that involved inherent vulnerabilities in designs and as-constructed conditions, subsequent chute slab deterioration, and poor foundation conditions in some locations. This incident was the result of interactions of numerous human and physical factors, beginning with the design of the project and continuing during the half-century until the incident, that is a long-term systemic failure. From independent forensic team: *“Although the practice of dam safety has certainly improved since the 1970s, the fact that this incident happened to the owner of the tallest dam in the United States, under regulation of a federal agency, with repeated evaluation by reputable outside consultants, in a state with a leading dam safety regulatory program, is a wake-up call for everyone involved in dam safety.”*

Mode of failure: Slab cracking and loss of under-drain system effectiveness; repeated repairs were ineffective and possibly detrimental. Progressive deterioration of concrete

and corrosion of steel reinforcing bars and anchors in the chute slab, with likely loss of slab strength and anchor capacity. Water injection through both cracks and joints, likely driven by stagnation pressure, resulted in uplift forces beneath the slab which exceeded the uplift resisting capacity and structural strength of the slab, at a location along the steep section of the chute. Shallow under-slab erosion due to rock quality, damage and deterioration of the underdrain system, and some loss of underdrain system effectiveness, which contributed to increased slab uplift forces. Poor foundation conditions likely contributed to low anchor capacity. The uplifted slab section (sudden failure), exposed the underlying rock at that location to unexpected severe erosion, resulting in removal of additional slab sections and more erosion. Emergency spillway's discharge eroded rapidly the rock at natural hillslope with head-cutting towards the emergency spillway. Operation of gates stopped flow over the emergency spillway but damages in the chute of service spillway worsened. A tremendous amount of debris went into the river channel. High waterways threatening to flood the powerhouse.

Rebuilt in 2017-2018.

- **Folsom Dam (USA,1995)**

Location: Sacramento, California, United States of America, on American River.

Dam: Combined Concrete Gravity and Embankment Dam, 103.7 m high.

Dates: Built in 1948-1956, incident in 1995. (Figure 3-147 and 3-148).

Spillway: Controlled by eight radial gates, five service radial gates 12.8 m x 16.15 m and three emergency gates, located on the gravity dam. Capacity 16,100 m<sup>3</sup>/s.

Incident: Reservoir was at full capacity. One of eight large spillway radial gates failed during reservoir releases on July 17, 1995. The gate failure occurred with a nearly full reservoir releasing a peak flow of about 1,130 m<sup>3</sup>/s. No injuries or fatalities occurred as a

result of the gate failure. Gate was being operated to maintain flow in the river during a power plant shutdown. The gate was opened following normal procedures; as the gate opening approached 73 cm, there was an “unusual vibration”, so gate hoist motor was stopped. When checking the gate, the right side of the gate swing open slowly, like a door hinged on the left side; water was pouring around both sides of the gate leaf. The time from vibration to observing gate displacement and uncontrolled flow of water was estimated to be no more than 5 seconds. Discharge was around 1,135 m<sup>3</sup>/s.

Mode of failure: Two main causes of the gate failure were identified: (1) Insufficient stiffness and strength in critical structural gate arm members and (2) Increased trunnion friction by corrosion of the steel trunnion pins. The friction was unaccounted for in the gate design. Due to the additional friction forces, the loads experienced by the trunnion pin increased loading in struts and braces of the gate. The resulting loads in these members exceeded the capacity of the strut-brace-connection bolts compromising the structural integrity of the gate. As the gate was operated, the failure initiated at a diagonal brace between the lowest and second lowest struts. Increasing corrosion at the pin-hub interface raised the coefficient of friction and, therefore, the bending stress in the strut and the axial force in the brace. The capacity of the brace connection was exceeded and it failed. This caused the load to redistribute and the failure progressed, eventually buckling the struts.

Lesson: Presence of a critical or fragile element, in this case the pin in trunnion with the added effect of poor maintenance. Had the trunnion pin been maintained with appropriate lubricant, protected from weathering effects, and inspected on a routine basis, the gate failure may have been avoided. A renewed focus was placed on maintenance and monitoring of radial gates, many of which were retrofitted to strengthen struts and bracing and ensure sufficient lubrication.

Rebuilt in 1996. In this dam a Risk Analysis resulted in the need to increase spill’s capacity. A new auxiliary spillway was designed and dam crest elevation was raised (2 m).

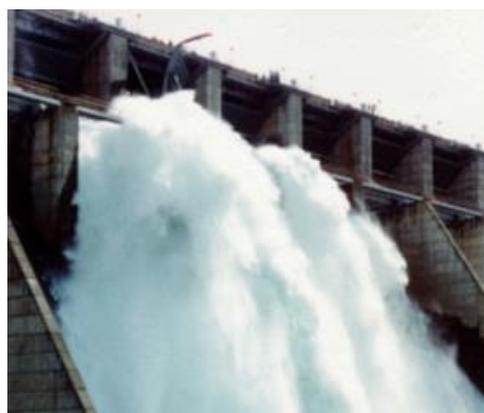


Figure 3-147: Failure of radial gate in service spillway. Folsom Dam, 104 m high (USBR, 1995)



Figure 3-148: Failure of a strut arm of a radial gate by impact

- **Machhu II Dam (India, 1979)**

Location: Gujarat, India, on Machhu River.

Dam: Embankment Dam, 25 m high.

Dates: Built in 1972, Incident in 1979.

Spillway: Controlled by radial gates. Capacity 5,670 m<sup>3</sup>/s

Incident: Flood events during monsoon storms are common in this region but in this case, it was larger than usual. The gates were opened except three on account of malfunctioning. Discharge was 5,555 m<sup>3</sup>/s very close to its full capacity of 5,670 m<sup>3</sup>/s, water con-

tinued to rise. Almost 24 hours later, embankment was overtopped on both sides of the masonry spillway leading to the failure.

Mode of failure: Flood was, at least 2.5 times the spillway capacity, with long duration and large volume. A hydrological analysis estimated a flood peak 16,307 m<sup>3</sup>/s.

Lesson: Incidence of uncertainty in hydrology in regions with extreme climate characteristics as monsoon storms.

Rebuilt in 1989 with a spillway capacity of 24,710m<sup>3</sup>/s.

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## Chapter 4. OUTLET WORKS

### 4.1 Overview

The outlet works are normally planned/ designed to operate for reservoir levels varying from Full Reservoir level (FRL) to Maximum Drawdown level (MDDL). They serve a variety of functions like providing water for irrigation, water supply, hydropower etc.

Besides they can also be used to assist in guaranteeing maintenance of minimum flows downstream in the water course/river on which the dam is built; they can also be used for evacuation of the sediments inside the reservoir and in some other cases, they contribute in the handling of extraordinary floods as along with the spillways to control the maximum water level of the reservoir.

From the point of view of hydraulic safety of dams, the outlet works associated with the hydroelectric, irrigation or water supply function do not assist in flood evacuation.

However, as stated in Chapter 1 of this Manual, *“dam safety encompasses all hydraulic works such as the reservoir/lake and its rim, the dam, the appurtenant works (spillways and outlet works), and structures close to the dam, watercourse/river, and abutments”*

On the other hand Hydraulic Safety of Dams encompasses both *“physical safety”* and *“operational safety”*, in the event of a flood. *Thus, the works, in this case - the dam and its appurtenant works, must retain their physical integrity, stability and resistance to safely withstand all the forces acting on it for all conditions of loading; and also, they must have reliable hydraulic performance. Since security means “almost at any moment”, safety should be verified in both normal and extreme conditions*

In general, the safety of an outlet works also depends on the type of dam; whether it is located in the body of an embankment dam,

they may have square external cross section and short length becoming more vulnerable and less in concrete or masonry dams.

For bottom outlets planned for flood evacuation the sluices have large dimensions. There are not many such outlets on large dimensions in India. A few examples are the sluices provided in Krishna Raja Sagar (KRS) dam, Karnataka (See Figure 4-1- ) and the under-sluices in Hirakud Dam, in Odisha which complement its surface spillway (DRIP, 2019) .

When they are used for other purposes, like irrigation, water supply etc. their size is not very large but should be adequate to comply with the functional requirements.

When outlets are located in an abutment, they are called tunnels, usually of much more length and, with circular or horseshoe cross section.

Much of the problems associated with erosion and piping in embankment dams equipped with outlets works have occurred at the contact of the conveyance structure of the outlet works with the embankment dam or in its abutments. For this reason, design of these works is not only required to take care of the functional requirements that this component must meet, but they must also be properly designed/detailed at the contact with the embankment dam so that its existence does not generate a fragile zone in the dam. (readers can also refer to the Manual on Assessing Structural Safety of Existing Dams, CWC 2020).

The biggest challenge to overcome in outlet works of Indian dams, and most of its components is aging. Taking as representative a sample of 3,000 dams in India, more than 67% of them are 30 years old or older and only 4% are less than 10 years old. (See Figure 4-2.)



Figure 4-1: Krishna Raja Sagara (KRS) Dam  
(<https://civilrmd.com>)

Age affects materials, operating mechanisms and surfaces, therefore, the probability of failure of the regulation component (gates and other associated hydro-mechanical equipment) or the deterioration of the surfaces due to abrasive is higher in older structures except for those dams with better maintenance/ supervision of their elements and in which rehabilitation, repair and/or replacement works have been carried out.

The electro-mechanical equipment is also affected by aging; there are some of them where its frequent use guarantees its operability while others are used periodically while some of these are only operated in extreme emergency conditions as responses to unusual events and their reliability can be very low.

The useful life of the gates, valves and other elements used for the regulation of the intakes is much less compared to the the rest of the civil components of the appurtenance works and for this reason, their replacement can become necessary when their physical condition or structural integrity becomes a requirement.

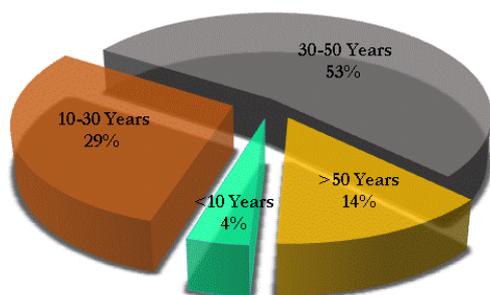


Figure 4-2: Age of India Dams (Adapted from CWC, 2019)

The outlet works are subjected to many and varied actions that gradually undermine their integrity and diminish their reliability. Among others, it is note worthy to mention the impacts that the solids transported by the water flows can produce, the blockage by sediments, debris or ice in its entrance section, processes of corrosion, abrasion or surface deterioration due to the high velocities, etc.

A clear example of the effect of the aging of control works on dams is the Krishna Raja Sagara (KRS) dam where a large rehabilitation project is currently under progress, framed within the DRIP project, in order to recover the operation of its outlet works/slucies.

KRS dam, built between 1911 and 1931, has a total of 169 sluices of which 152 are dedicated to the management of floodwaters and the other 17 are used for irrigation and maintenance of life in the river and other secondary uses.

According to the studies carried out between 2015 and 2016, the gates of only 72 of the 152 sluices for the control of floods were in working condition, reducing the discharge capacity of the dam by 32% and, thus, decreasing the hydraulic safety of this important reservoir.(See figure 4-3)

The replacement of 152 gates and updating and modernization of the operation of these gates is being carried out under DRIP Project; this will help in improving the capacity of the dam to attenuate the extraordinary floods, as well as prolonging its useful life and reducing the risk levels of both the dam and the population located in the downstream areas.

The objective of this chapter is to provide information to assess whether the intake and outlet works have the levels of hydraulic safety that are considered appropriate as per present day standards, verifying their potential failure modes, evaluating consequences and assist in working out the

rehabilitation measures that could be taken up to ensure their safety.



Figure 4-3: Condition of Gates in KRS Dam (CWC; 2018)

To facilitate the understanding of the hydraulic safety assessment process, frequent failure modes have been defined in each of the components of the outlet work, the causes that could lead to their occurrence and the features that must be incorporated to prevent failure.

**REFER APPENDIX A**

For Common Failure Modes linked to Outlet Works , see FM-16 to FM-23)

For the design of any particular structure, the Standards developed by the Bureau of Indian Standards and specialized technical literature developed by institutions such as

ICOLD, USBR, USACE, FEMA can be used and, thus, the design can be reviewed adopting a criteria accepted internationally.

**4.1.1 Definition and Function**

The most common definitions of Outlet works are:

**FEMA:** “A dam appurtenance that provides release of water (generally controlled )from a reservoir.”

**USBR:** “A combination of structures and equipment required for the safe operation and control of water released from a reservoir to serve various purposes”.

“A series of components located in a dam through which normal releases from the reservoir are made. A device to provide controlled releases from a reservoir. A pipe that lets water out of a reservoir, mainly to supply downstream demands.”

Specifically, sluices have been defined by the Bureau of Indian Standards in the Glossary of Terms relating to river valley projects - Part 8 Dams and Dam Sections (IS 4410 Part 8) as: “A conduit, fitted with a gate, for carrying water at high velocity”.

For the purposes of this Manual, it is considered that Outlet Works are hydraulic structures that allow the use of reservoir waters for various purposes for which the dam has been planned/constructed.

The functions for which the outlet works are provided are as follows:

1. **Flood control:** They are the outlet works designed with greater capacity of conduction. From the point of view of Hydraulic Safety, they are clearly of utmost importance since their malfunction will lead to increases in the reservoir levels, affecting the hydraulic safety issues. In these outlets, the controls are usually exercised by gates.(See figures 4-4 and 4-5)



Figure 4-4: Under-Sluices in Hirakud dam (DRIP, 2019)

Since these outlets carry large flows with high velocity they are prone to significant cavitation and abrasion problems. These structures perform are designed to operate under free or pressure flow regime, as the project dictates. The waters are usually discharged into the river on the downstream side.



Figure 4-5: Flood control at Krishna Raja Sagara Dam (www.thehansindia.com)

- Irrigation Intakes:** They are much smaller intake structures than the used for flood. Usually water is discharged into an irrigation canal that directs the flow to cultivated land. Sometimes the flows are discharged in the river itself and, subsequently, are captured by diversion works. Its contribution in the Hydraulic Safety of the dam is low. The malfunction of the water supply for irrigation compromises the delivery of the water to the areas benefited from the dam (See figure 4-6 and 4-7).

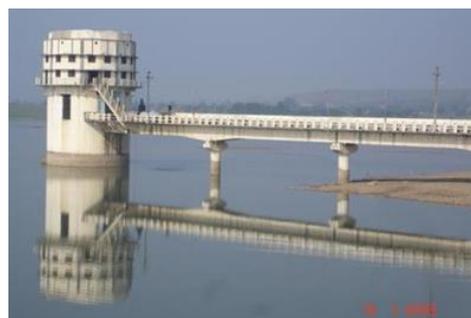


Figure 4-6: Intake tower Rajghat Dam (Sagar Municipal Corp; 2009)

- Hydropower Intakes:** The outlet works associated with hydroelectric projects i.e. tunnels / penstocks convey water to the powerhouse. After the waters are used to run turbines and generate hydropower, they are returned back to the original watercourse. The malfunction of these intakes compromises the safety of the primary function of the reservoir but does not usually affect the Hydraulic Safety of the dam. (See Figure 4-8)



Figure 4-7: Neyyar Dam (Kerala Irrigation Department)

The velocity of flow in this type of intake is normally low in order to minimize energy losses between the reservoir and the turbine entrance and therefore, its cross section can be large and the hydraulic problems are limited. At the entrance of these intakes there are usually trash-racks and other elements that prevent the entry of large solids into the conduction structure that could affect the rotating components of the turbines.



Figure 4-8: Penstock (8 no.) to Balimela Power House - Machkund-Sileru River (Odisha Hydro Power Corporation Ltd)

4. **Water Supply:** They are usually the most complex waterworks since the water withdrawal is done from a vertical intake structure that facilitates the selective intake of water from the depth considered to be the best from considerations of water. They are structures with low discharge capacities and normally have two (2) hydraulic controls. The first one allows to select the level from which water is drawn while the second regulates the amount of water drawn. As it is an outlet for water supply to the population, keeping the system operational is of the utmost importance, and therefore, it is usual to have more than one control equipment capable of keeping the intake operational even in unusual circumstances. The waters captured by these intakes are usually taken under pressure to the Water Treatment Plants and from there to the population served. (See Figure 4-9).



Figure 4-9: Tower Intake Mukkadal Dam (CWC)

5. **Low-Flow Requirements:** The basic function of outlets catering to this requirement is to preserve wildlife, maintain flows that sustain adequate environmental conditions. The construction of a dam entails a total modification of the runoff conditions in the downstream river stretch and, therefore, affects the aquatic fauna and, in general, of all the wildlife associated with this watercourse. This structure does not contribute to the Hydraulic Safety of the reservoir and its malfunction is usually taken care of by operating any other outlet of the reservoir. (See Figure 4-10)



Figure 4-10: Cataract Dam outlet works discharging low flow requirements Dam (Water NSW)

6. **Sediments:** Proper sediment management is one of the most important challenges for the sustainability of reservoirs. The accumulation of sediments in the reservoir is inevitable. The accumulation obstructs the intakes due to the large concentration of suspended solids. Usually the inlets that handle the sediments are designed for emptying the reservoir and, therefore, are large

structures, capable of discharging large flows at high velocities and with high concentrations of solids. (See figure 4-11 and 4-12)

7. **Drawdown of the reservoir** : These outlets are designed with the objective of allowing partial or total drawdown of the reservoir. The reasons that justify the



Figure 4-11: Sediment removing at Jiroft Dam – Iran(www.entura.com.au)

The function of this outlet is directly related to Hydraulic Safety and therefore it is necessary that its operation be safe and reliable. In some cases, the outlets are used to maintain the reservoir levels and, therefore, can be of great support when contributing to the routing of extraordinary floods. Due to their location (usually near the bottom of the reservoir) these structures can also be used as a river's management structure during the construction of the dam and the structure that regulates the first filling of the reservoir.

Any hydraulic system may have more than one outlet works and each of them is planned for a specific purpose and, in some cases, when as bottom outlets, its function can vary over time throughout its service life, a situation that makes its design and operation complex.

#### 4.1.2 Classification

Outlet works can be classified in different ways:

- By the use or purpose as stated in the previous paragraphs

partial drawdown of the reservoir are many: repairs of some component, maintenance, inspection of the condition of the dam or some of its appurtenant works, among others. The total drawdown may be associated with the failure of some component of the dam that compromises the overall safety of the dam.



Figure 4-12: Shihmen Dam – Taiwan (Peellden; 2017)

- By the relative location within the dam

The Bureau of India Standards classify them

In terms of the Head above the intake for hydropower use, measured from the FLR to the intake center line (IS9761).

According to the head above central line of intake for a reservoir type of intake Low Head: Up to 15 meters

- Medium Head: From 15 to 30 meters
- High Head: Above 30 meters

According to the flow type of operation ~~its~~ the following classes could be identified where the location of the control mechanism (valve or gate) is the one that defines its condition:

- **Free surface flow**, normally the hydraulic control structure is located at the upstream end of the outlet
- **Pressure flow**, when the control element is located at the downstream end of the outlet.

- **Mixed flow** when the control structure is located in an intermediate place of the discharge line that allows the tunnel or pipe to be pressurized before the regulation structure and free surface open channel flow downstream of it.

According to its location within the dam:

- Surface intake,
- Bottom intake
- Intermediate intake

According to the characteristics of the Intake:

- Submerged intake
- Intake Tower or Multilevel Intake

According to its discharging point/location:

- River outlet
- Canal Outlet

### 4.1.3 Components

Figure 4-13 and 4-14 show profiles of outlet works. The elements of the outlet are the following:

**Inlet Structure or Intake:** It is the structure from where the water contained in the reservoir enters the outlet work. It can be a submerged structure or reached from the dam top under all conditions. It may be located in the dam or upstream of the dam.

In the case of hydropower intakes, trash rack structure is provided at the entrance of Power intakes for protection against floating

material and objectionable debris entering the penstock/water conductor system.

**Control Devices:** They are the mechanical equipment used to operate the outlet works. They consist of gates or valves that control the amount of flow required to be passed. They should be able to operate under both fully and partially open conditions.

**Conveyance structure:** It refers to the element used to carry the water from the intake to its discharging point or to the channel/conduit that will ultimately carry the water to the proposed location. The conveyance structure can take different names depending on the type of dam and where it is located. They can be as follows :

- Called "Sluice" when the conduit crosses the cross section of a concrete dam or masonry dam,
- "Conduits" when they cross or are under an embankment dam and,
- "Tunnels" when they are excavated in the natural abutments of the dam.

**Terminal structure:** This is normally a stilling basin or any other energy dissipation device that dissipates the energy of the flowing water before it is delivered to the receiving watercourse.

All the outlet works have these four components although in some of them the formal differentiation between one and the other is not appreciable.

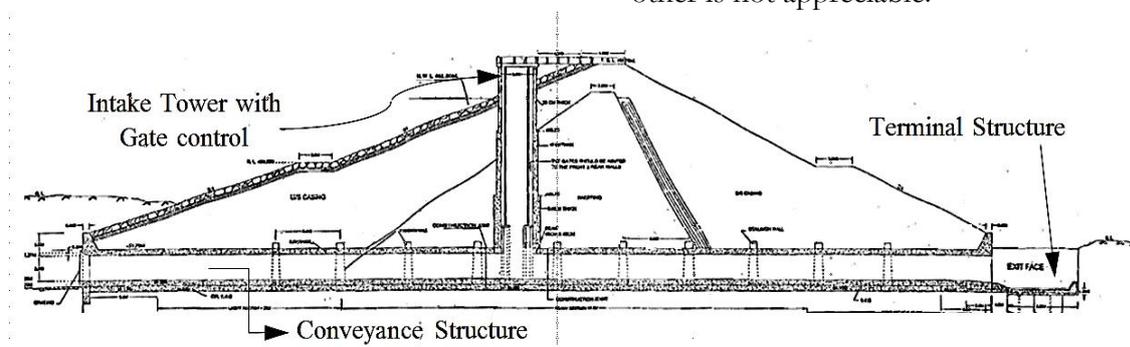


Figure 4-13: Profile of outlet works in Embankment dam

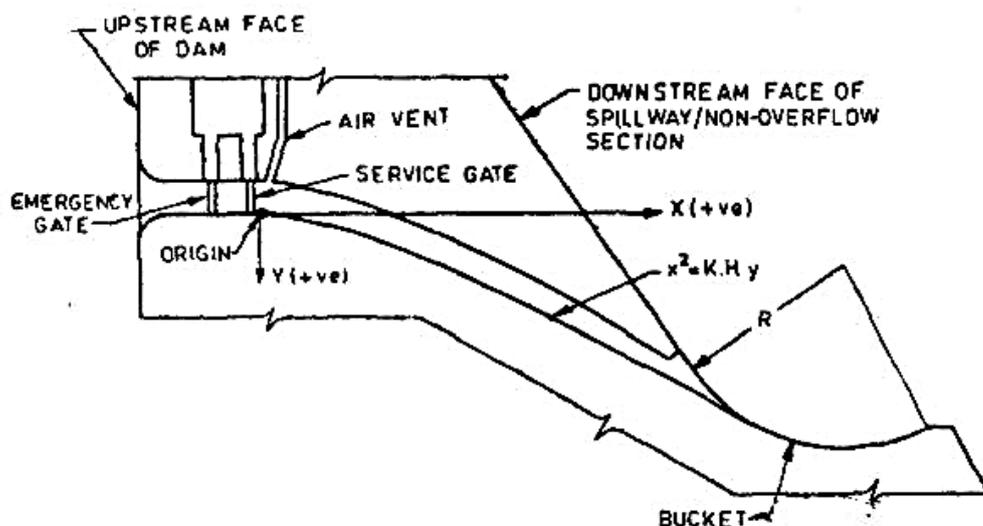


Figure 4-14: Sluice profile (IS-11485.1985)

As mentioned, the dimensions of the outlet and characteristics of the flow will vary according to the reservoir elevations and the flow discharge to be passed. The pressures and the velocities to which the various elements of the outlet works will be subject during its normal and extraordinary conditions of operation, are of primary concern during the design stage. Also, the magnitude of the flow that pass through the outlets and the abrasion and / or cavitation processes that can potentially occur in them must be given special attention in the designs and operation and their effects can be observed by inspections during the operation of the structure.

## 4.2 Description of Outlet Works

### 4.2.1 Intake structure

The intake is the initial component of the outlet work and its function is to capture the water stored in the reservoir and transport it to the conveyance structure with the minimum possible energy loss, avoiding the formation of vortices and trash free.

The introduction of the intake is easier in masonry and concrete dams than in embankment dams. In concrete dams the intakes are usually openings that are left in the body of the dam.

The location of this structure depends on its function, site conditions, project requirements, etc. Mostly the intake works are located in the lower third portion of the reservoir since it is in this sector that the use of most of the useful volume of reservoir is guaranteed taking into account the yearly reservoir level variations from FRL to MDDL.

As a general criteria, the intakes of large size outlets and surface spillways which are intended to assist in the evacuation of flood or for the control of reservoir levels are usually located at intermediate or higher levels of the reservoir.

The location of the intake, in the case of hydroelectric projects maybe adjacent to the dam or u/s of the dam depending on the project layout.

Intakes for sediment evacuation will normally be close to the original riverbed and should have large size outlets which may also help to supplement the spillway in the case of the outlet adding capacity to the spillway discharge during large floods. Sluice gates at intake of KRS dam in Karnataka may be seen in Figure 4-15.



Figure 4-15: Gates in Sluice Intake at KRS, Karnataka (CWC; 2018)

In embankment dams the location of the intake structure can be located in the dam or upstream of the dam in one of the abutments ( Figure 4-16 and 4-17).

At the entrance section, a smooth transition is normally designed so the water flow accepts a minimum disturbance reducing the possibility of formation of secondary flows and vortices that affect the efficiency of the intake.



Figure 4-16: Coquitlam Tower Cherry Creek Dam outlet works (www.mcelhanney.com)



Figure 4-17: Varattupalam Dam. (CWC; 2017)

**REFER TO APPENDIX C**

Related to Boom barriers to retain floating debris

In locations where there is a lot of floating material and debris it is necessary to install trashracks that prevent materials from entering the outlet. Trash racks - are of vital importance in intakes for hydroelectric plants, water supply systems and some irrigation systems. ( Figure 4-20). In an inlet is designed for other uses such as flood releases, or large flow discharges, the trashrack is not used.

For hydropower plants, trash racks are designed for a velocity of 0.75 to 1 m/s, assuming that 50% of the effective area is obstructed although this limit can be modified in those cases where there are mechanical rakes that facilitate the removal of floating material/debris trapped by the trash racks. Racks have to be cleaned either manually or by trash rack raking machines on regular basis. Normally the separation between bars is in a range of 40 to 100 mm.

In surface structures or near the surface, the probability of obstruction with floating material/debris such as logs and ice may be likely depending on site conditions; so floating barriers that impede the passage of floating material/debris towards the intake or relief structures may be necessary.

The efficiency of the intake works can also be affected by the formation of vortices that incorporate air into the water flow. This incorporation of air impacts mostly in hydroelectric works where the loss of efficiency in turbines alters the production of energy and the economy of the project.

The formation of vortices depends on many factors, which include the submergence of the inlet, the shape of the entrance, the conditions of flow, the effect of other structures located nearby and the shape, location of the trash racks and approach flow conditions to the intake.

Predicting the formation of vortices particularly when dealing with 3D conditions is complex. Physical models can be used to

evaluate/visualize their formation and the causes that generate it with sufficient precision.

**REFER APPENDIX D**

Where the utility of physical and mathematical models in the prediction of the behavior of hydraulic structures is analyzed

The most important variable affecting vortex formation is submergence; which can be critical during the dry season when the reservoir is at the minimum levels.

**Types of Intake**

Intakes of different configurations include the following:

**Intake Structures for Concrete/Masonry Dams - Sluices**

In sluices, the intake structure is generally a part of the body of the dam..

Depending on the function for which they are planned, the sluices can be founded at different levels of the dam. They can be founded near the deepest level of the reservoir in case they are to be used for river diversion operations during construction or for emptying the reservoir in an emergency or, for flushing sediments. When its primary function is the management of reservoir levels for evacuation of floods, the sluices can be founded near the ground surface or at some intermediate levels. (See figure 4-18)

The invert level of the outlet is generally kept above the new zero elevation of the reservoir, the new zero being the estimated level that the accumulated sediments will have in the expected time of operation of the reservoir determined based on the prediction methods of sediment distribution in the reservoir. reservoir according to standard IS 5477-2 Fixing the capacity of reservoirs. Part 2 - Dead Storage



Figure 4-18: Sluice Intake at KRS Dam (CWC;2018)

**Drop Inlets / Riser**

They are structures that capture the water in the reservoir when it exceeds a certain level. They can have trash racks that cover the entrance section and their function is usually complementary to relief works for passing extraordinary floods.(See Figure 4-19)



Figure 4-19: Typical riser structure (ASDSO)

**Submerged Intake Structures**

In those reservoirs where large amounts of floating matter, debris and ice are expected, the submerged structures prove to be much less vulnerable and, are therefore, preferable to the installation of surface intakes.

Most of the bottom outlets are at or near the bottom of the reservoir; these structures however may get obstructed by sediments that accumulate at the bottom of the reservoirs.

The intake, in addition to having the flared entrance section, includes protection trash racks and emergency gates that allow the emptying of the conduction structure for inspection and maintenance.

The obstruction of trash racks at the entrance in a submerged intake can lead to operational problems and, in some cases, preventing the delivery of water for the intended use (See Figure 4-20).



Figure 4-20: Pathfinder Dam, Trashracks at entrance to Pathfinder tunnel(Tango images/Unknown)

### Tower Intake Structures

The Towers Intakes are, perhaps, the most complex intake structures because of their size and large amount of controls and protection elements they feature.

They are designed to draw waters from different levels that have better physical-chemical characteristics for water supply to the population. However, towers are also provided where it is planned to operate the outlet for irrigation with either control gates without the limitations of a submerged outlet. (See Figures 4-21 and 4-22).

The structure of the towers is vertical and to access them it is necessary to have a bridge that links the operating platform with the crest of the dam. In the case of towers adjacent to concrete or masonry dams, the access, foundation and operational problems are considerably reduced.

Internally the tower can be dry or wet depending on the actuator mechanisms in the control elements.



Figure 4-21: Tower Intake Varattupalam Dam

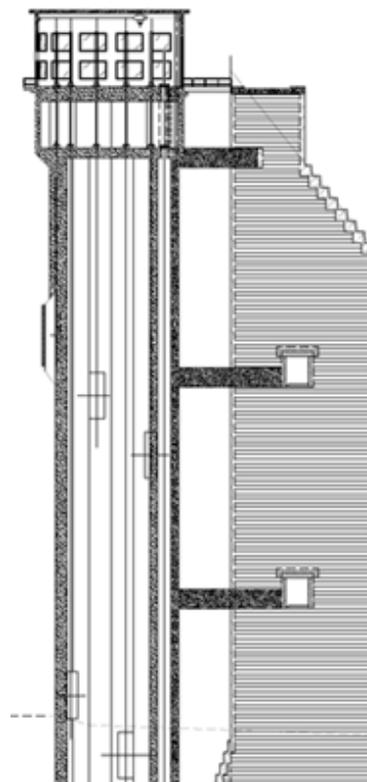


Figure 4-22: Tower Intake Drawing for Cuira Dam (Venezuela) with 8 inlet levels (Hidroven; 2010)

The number of inlet levels of the towers for water supply depends on the depth of water in the reservoir although the minimum number of levels in a multi-level intake is considered 3.

### Inclined Intake Structures

The inclined intake structures are a variation of the tower intakes. They rest on a competent surface to receive the stresses produced by the weight of the structure and its operating loads. In concrete dams that support surface can be the upstream face of the dam. (Figure 4-23).

The inclined intake can be an alternative in embankment dams if it is desired to capture water at various levels and also in a strong natural abutment in which, there exists a tunnel through which the river was earlier diverted during the construction of the dam (Figure 4-24).

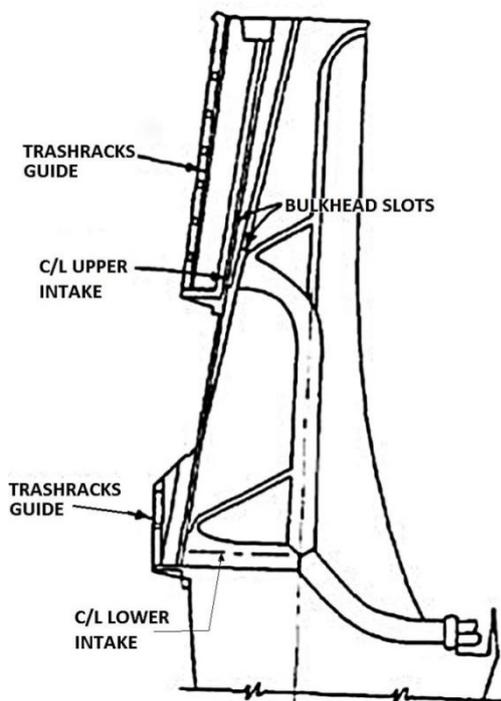


Figure 4-23: Intake on face of Dam  
(<http://ndl.ethernet.edu.et/>)



Figure 4-24: Inclined Intake Tower Nichols Reservoir, USA. (Albuquerque Journal; 2014)

### Auxiliary Intakes

The various functions that an outlet can perform during its useful life may involve either an independent design for each specific function or, in some cases, through a multifunctional design by combining it with another outlet built earlier for some other purpose.

Thus, an intake structure originally intended for river diversion during construction can be re-used by making necessary transformations in the intake structure that allows sediment evacuation, reservoir emptying and, adapting an auxiliary intake, or for maintaining the minimum required releases for the conservation of the fauna and flora of the riverbed.

These auxiliary inlets, although smaller in size than the main intake, may require trash racks (that prevent the passage of floating matter/debris transported by water), emergency gates and stop logs for closing in case of emergencies, the flared entrance that reduces losses and other valves or service gates that allow to regulate the discharge.

### 4.2.2 Control Devices

The control devices are required in outlet works for regulating the discharge or some for some other purpose like isolation of a certain reach etc. The appropriate selection of the control device (gate/valve) taking into account the working conditions will result in

the reliability, safety and effectiveness of operation.

The control devices can vary considerably although, from the hydraulic point of view, they can be classified as Gates and Valves. Some of the gates are of fully open/fully closed type and some of them are of regulation type.

There are numerous criteria for classification; IS: 13623: 1993 “Criteria for choice of gates and hoists” may be seen in this regard in which they are classified taking into account mechanical, electrical or other aspects.

From the hydraulic point of view the most relevant aspects to define the classification of the control devices are:

According to the head above the Centre line of the opening at intake in Reservoir type Hydro-Power Intakes (IS 9761):

- Low head: Less than 15 m.
- Medium head: Between 15 and 30 m.
- High head: More than 30m

According to its location relative to the reservoir level:

- Crest type.
- Submerged.

According to the mode of operation:

- Regulatory: Allows partially open operation.
- Non-regulatory: They work as fully open or closed.

According to the operational requirement:

- Service: It is the main control element (Gate/Valve) used to regulate the flow in the intake work.
- Maintenance: Devices installed in order to isolate the main service element and perform maintenance, inspection or replacement.

- Emergency: Device used to close the outlet when there is an event that may compromise the safety of any of the components located downstream.
- Construction: Closing elements of the control or diversion structures of the river used during the construction stage.

In general, the devices related to construction, emergency and maintenance are gates due to their greater robustness, ease of operation and space required for their installation; however, the service devices can be valves or gates depending on the flow, pressures and location of the control element.

In multi-level intakes it is common to have two (2) control elements, the first one located in the intake, defines the area of the reservoir from which the water is extracted and the second, located anywhere in the conduit, for backup/maintenance purposes.

There are a wide variety of gates and valves, many of them having wide application, however, very specific in their use. There are some types of control devices that, due to their complexity or due to their unsatisfactory experience in other projects have now become obsolete.

A brief summary of the most commonly used types of control elements follows:

### **Gates**

The operation of the gates can be mechanical, electrical or hydraulic which will depend on the type of gate and the loads/thrust.

They can be divided into two (2) main groups: flat gates and radial gates.

The flat gates, where the vertical gates stand out have a great variety depending on some attachments that are incorporated to facilitate their placement, mobilization or

resistance to the actions to which they will be subjected.

Vertical gates are both regulatory and non-regulatory.

Within them we can highlight the following type of gates:

**Stoplog:** These are the most commonly used gates for the closure of an intake work when an emergency occurs or when it is desired to maintain / repair the service gate or for inspection of the structure. Normally they are located at the upstream end of the intake and are placed in quite water; sometimes even before the inlet mouth. Being an emergency device and located at the upstream end of the duct, it is common to have a few set of stop log gate units for several ducts which will be inserted in guides designed for this purpose at the entrance section.(See Figure 4-27)

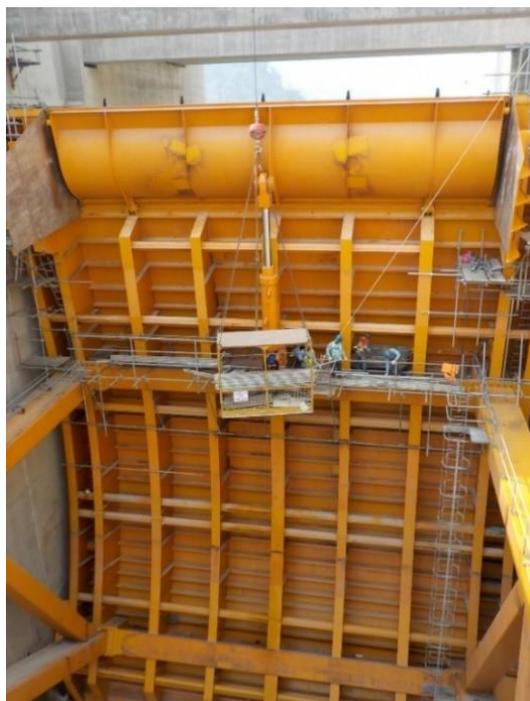


Figure 4-25: 19x24.5 m Radial gate with flap. Xayabury Dam- Laos (KGAL Consulting)

**Vertical lift/slide gates:** These gates slide along guides vertically until the water flow is completely interrupted. They are devices used for maintenance and emergency work located just upstream of the service device.

Because of their robustness they allow for their opening and closing even in adverse conditions and are very reliable. Wheels that reduce friction stresses between the gate and its movement guide are generally incorporated into the gate to reduce the power of the equipment needed for operation. (See Figure 4-26)



Figure 4-26: Vertical Lift Gates at Leslie Harrison Dam (Australia) (Shree Shakti Eng.)

**Radial or Tainter:** In these gates the leaf that faces the flow is a circular sector that allows to combine structural efficiency in resisting the thrust to which it will be subjected with hydraulic efficiency by developing a much more hydrodynamic flow transition than that in a flat gate.

The radial gates are increasingly used because compared with the flat ones they are lighter, require less operational effort and generate less hydraulic inconvenience as the incorporation of lateral guides which is not necessary. They are used as regulatory or service gates and can be placed in the free surface or submerged. These days in almost all projects, the gates located on the crest of the spillway are radial due to their comparative advantages and high degree of reliability. (See Figure 4-25 and 4-28)



Figure 4-27: Stoplog gates (Shree Shakti Eng.)



Figure 4-28: Tainter Gate at Spillway (Steel Fab Inc.)

REFER TO APPENDIX E  
Operational Safety of  
Hydromechanical Equipment

### Valves

Valves, on the other hand, are control mechanisms that remain within the flow at all times and are normally service devices capable of controlling the flow and, serving as energy dissipators.

There are a wide variety of valves, many of which are described in Indian standard IS: 4410 (Part XVI/Sec2) -1981 “Glossary of terms relating to river valley projects. Part



Figure 4-29: B A Hilton Throttling Knife Gate Valve at a California dam project (Hilton, 2015)

XVI Gates and Valves. Section 2 Valves” some of the most used are described below.

**Gate Valves:** Gate valves or sliding valve gate are used as maintenance and emergency valves and in some cases as regulation. Its design must be done so that they can be operated in conditions of unbalanced load and maximum flow.(See Figure 4-29)

**Butterfly valves:** Due to their structure, where the closing blade is immersed in the flow, they are devices that are used to limit the speed to 10 m/s (30 fps) so that there are not too many problems of flow separation and cavitation. They are used as maintenance and emergency valves but are not usually used as regulating valves. (See figure 4-30)



Figure 4-30: Butterfly Valve (FNencini SRL)

**Hollow Jet valve:** This is one where a central needle-shaped device moves in the longitudinal direction of the valve induces the flow to separate and discharge in the form of a hollow jet. (See figure 4-31)



Figure 4-31: Hollow Jet Valves at Rur Dam Germany (VAG, 2016)

**Howell Bunger Valve:** they are structures installed in such a way that they discharge outdoors or at atmospheric pressure. The

discharge is released as a hollow jet although in this case the sealing element divides the flow in a conical manner facilitating the loss of energy from the flow. Sometimes and where the conical discharge of the jet is inconvenient, the deflectors that direct the jet to the area destined for reception are installed at the outlet. They are regulating devices for which they are designed to withstand high pressures, partial openings and high velocities. (See Figure 4-32)



Figure 4-32: Howell Bungler Valve at Beleña Dam, Spain. (2013)

**Needle Valve:** They are similar to hollow jet valves where the conical needle-shaped shutter device moves longitudinally keeping the face facing the flow still. (See Figure 4-33)



Figure 4-33: Needle valve 117” diameter San Gabriel Dam (J. Husted; 2016)

### Control chamber

The elements used for operation of the control devices are housed in a chamber called the Control Chamber.

The control chambers are required for both the service or regulation device and maintenance device that allows the flow to be closed during the inspection, repair or replacement of the service gate or valve.

It must be easily accessible under any situation that occurs in the reservoir. The review or design of the control chamber is beyond the scope of this Manual; However, it is prudent to clarify that all control elements must be continuously monitored, maintained and supervised in order to use them during routine normal operations or during emergency events or for the work of maintenance.

The inability to operate any of the emergency or service devices may imply inability to use the outlet and the loss of one of the components of the dam and, of course, the function/benefits for which it has been designed.

### 4.2.3 Conveyance structure

The conveyance structure is the element that allows the water drawn from the intake to be conveyed to the place of its final disposal, whether it is through the energy dissipaters, and then to the river or, to a channel or, is directed to another structure that transports

it to the site of its use. The planning and design of a sluice located in a concrete or masonry dam, an outlet embedded in an earthen dam and a tunnel located in the abutment upstream of the dam/reservoir rim are all totally different from each other.

Sluices in a Masonry/Concrete dam are usually short in length and rectangular in section and of various dimensions. In embankment dams, however, they tend to be much longer with a circular or horseshoe or rectangular shape (with/without collars) and changes in their shape/transitions in the control chamber or in its terminal section.

### Sluices

Sluices are conduits that are built in the body of a Concrete or Masonry dam. From the structural point of view, they constitute an opening in the cross section of the dam.

In many cases they are located in the spillway section depending on project planning. This kind of a layout, where ever possible, avoids the need of a separate energy dissipator.

However, there are limitations in operation especially if these sluices are to be used for passing the flood discharges in conjunction with the surface spillway. An example of the same is in Hirakud dam, Odisha where the large size under sluices are to be operated first and the surface spillway thereafter.

Table 4-1 summarizes dimensions of sluices provided in some Indian dams, their discharge capacity and a comparison with flow through the spillway of the same project.

As it can be seen in the table, most sluices have relatively small dimensions although due to their location, usually in the lower area of the reservoir, they have relatively high velocities of flow.

Most of these small size sluices have virtually no influence in augmenting the spillway capacity and their dimensions are only sufficient to enable them to meet requirements for irrigation, water supply etc.

Name of Dam	Function	Number of Sluices	Sluice Characteristics				Spillway Characteristics		Q Sluice vs Q Spillway
			Size of Sluice		Discharge Capacity		No. of Bays	Discharge Capacity (m <sup>3</sup> /s)	
			Width (m)	Height (m)	Each (m <sup>3</sup> /s)	Total (m <sup>3</sup> /s)			
Bennithora	I	2	2.00	1.40	4.05	8.10	7	12,176.12	0.07 %
Lower Mul-lamari	I, H, WS	2	1.40	1.85	8.51	17.02	5	3,189.00	0.53 %
Tawa	I	4	1.83	2.43	30.01	120.04	13	20,500.00	0.59 %
Chikkahole	I, WS	2	1.82	0.91	2.27	4.53	4	765.00	0.59 %
Nayayanpur		4	1.50	2.50	71.00	284.00	30	37,945.00	0.75 %
Almatti	I, H	6	1.50	3.10	56.63	339.80	26	31,007.00	1.10 %
Kuttiyadi	I	1	Diameter	1.20	18.00	18.00	4	1,444.00	1.25 %
Balimela	I, H	2	2.44	3.66	113.50	227.00	10	10,930.00	2.08 %
Panchet	H, WS, FP	10	1.73	3.05	98.00	980.00	16	17,840.00	5.49 %
Malaghatta	I, WS	2	1.22	1.83	16.95	33.90	1	344.90	9.83 %
Mettur	I, H, WS	13				1571.55	16	12,513.29	12.56 %
		5	2.13	4.27	84.95	424.75			
		8	3.20	4.88	143.35	1146.80			
Krishna Raja Sagara	I, H	152				9886.60			
	Spillway Sluice	48	3.05	3.05	26.84	1288.53			
	Spillway Sluice	40	2.44	3.66	56.43	2257.36			
	Spillway Sluice	48	3.05	2.44	60.34	2896.47			
	Spillway Sluice	16	3.05	6.10	215.26	3444.23			
	Irrigation Sluice	3	1.83	3.66	78.10	234.29			

Name of Dam	Function	Number of Sluices	Sluice Characteristics				Spillway Characteristics		Q Sluice vs Q Spillway
			Size of Sluice		Discharge Capacity		No. of Bays	Discharge Capacity (m <sup>3</sup> /s)	
			Width (m)	Height (m)	Each (m <sup>3</sup> /s)	Total (m <sup>3</sup> /s)			
Irrigation Sluice		1	1.83	2.44	7.08	7.08			
Irrigation Sluice		2	3.05	3.05	22.65	45.31			
River Sluice		3	1.83	4.57	106.75	320.26			
Scouring Sluice		8	1.83	3.66	106.56	852.45			

Table 4-1: Details of Sluices provided in some India Dams (Adapted from CWC; 2020)

However, there are some important exceptions in this list in respect of those dams in which a large number of sluices of relatively large size have been provided which serve as a spillway for passing the floods.

Krishna Raja Sagar dam in Karnataka with its 169 sluices is one of the most important example of the use of sluices in India for passing the flood besides Hirakud dam in Odisha. A total of 152 of its 169 sluices are planned to pass the floods. The hydraulic safety of the dam is therefore directly related to the operation of the gates and other control equipment in these 152 sluices. The remaining 17 sluices provide for irrigation requirements, maintenance of minimal environmental flows in the river, removal of sediment etc.

Most of the evaluated outlets have a relatively low discharge capacity when compared to the discharge capacity of the spillways (less than 10%), therefore, their contribution at the time of passing extraordinary floods is not significant.

However, the sluices can be very helpful if they are designed to keep the reservoir levels low before the arrival of the flood and facilitate in the passage of extraordinary flood along with the main relief structures (early releases).

The sluices dimensioning depends on the expected function to be performed. For instances, if these sluices are to be designed to empty the reservoir, the dimensions should be such that they can empty the

reservoir at a given number of days to a minimum safe reservoir level.

In case the sluices are to be planned for removing sediments from the reservoir, the dimensions should be such so as to mobilize the sediments, and maintain density currents for keeping the sediments in suspension for the eventual expulsion from the reservoir.

Those sluices responsible for maintaining controlled reservoir levels (flood management) should take into account the flows of the monsoon season and the capacity of the reservoir to handle the associated floods.

Other uses of these sluices, such as for hydroelectric uses, irrigation and domestic /industrial water supply have specific design conditions that result in specific dimensions and velocities that must be taken into account when designing the conveyance structures.

### Conduits and Tunnels

The conduits that cross the embankment dams as well as the tunnels that are excavated in the abutments of the dam or on another hillside around the reservoir rim, are designed considering two fundamental aspects: to preserve at all times the safety of the dam and to design structures capable of performing the expected function in the most efficient and economical way.

These types of works are rarely intended to be used for passing the floods except in some special projects where they

supplement the spillway to pass the floods or for the reduction of reservoir levels just before the occurrence of an extraordinary event.

The planning of outlet works in Earth dams must consider not only the hydraulic and structural design associated with the structure but, in addition, it must incorporate other aspects related to where it will be founded, and the measures that have to be taken in order to avoid leaks, settlements, among others.

Physically, the conduit that crosses an embankment dam is a discontinuity in the contact of the landfill with the foundation. Thus, in a cross section of Embankment dam, the conduit represents a support of the fill above it, but of a better quality than the adjacent land and, therefore, there is a possibility of potential differential settlement and the eventual formation of fissures and cracks in the embankment. Likewise, the longitudinal profile of the conduit modifies the load distribution below the conduit on account of load of the dam body that it supports and, more important, the foundation conditions also vary along the conduit which may cause differential settlements along the conduit itself.

Construction defects, inadequate compaction of the earth fill etc. at the contact between the dam and the conduit/outlet may result in leakages/preferential waterways that can lead to initiation of erosive processes, piping, etc.

Nearly 25% of the in embankment dams have some relation with erosive processes initiated as a result of the conduits that cross the body of the dam. By understanding the processes it is possible to establish designs, construction methodologies and supervision, monitoring and inspection systems that result in the overall safety of the dam and reduce the failure of dams due to these reasons.

### Classification by flow regime

Hydraulically the conduits can be classified according to the flow regime that occurs in them. There can be three types of flow :

- **Free flow:** In free flow the conduit carries water at atmospheric pressure at its surface (open channel flow). The hydraulic control is at the entrance of the outlet works.
- **Pressure flow:** In these cases the flow remains pressurized throughout the length until reaching the control structure that is located at the terminal location of the conduit/outlet just before the energy dissipater.
- **Mixed flow;** This occurs in conduits/outlets where the hydraulic control structure is located at an intermediate point along its length. In this case, pressure flow occurs upstream of the control structure and free surface flow downstream of it.

As can be seen, the flow established within the conduit is defined at the time of deciding the location of the control structure and, depending on its location, design strategies can then be planned to reduce problems associated with dam safety.

### Free flow

The conduits operating in free flow condition are those where the control structure is located upstream; that is to say that at the entrance section the outlet is submerged and in contact with.

By having a closing element (Gate) at the entrance, it is possible to inspect the entire duct entering from the downstream end and to evaluate its integrity, deterioration and, eventually, make repairs as required.

Under these loading conditions, the loss of water tightness of the conduit (on account of cracks in the ducts which may develop with time or due to poor quality of

construction) can imply entry of water into the conduit (seepage) that could involve migration of soil particles from the Earth dam and generation of a preferred water route that compromises the safety of the Embankment dam.

### **Pressure flow**

The outlet works that operate fully under pressure have a control structure/control device just before their terminal point where the water is discharged in to the energy dissipator in the receiving channel.

Access to the control structure is very simple since the control chamber is located at the downstream of the dam; however, the internal inspection of the outlet is hindered by the pressure flow that occurs throughout its length and only by installing an emergency gate or stop log at the entrance section can the outlet be emptied in order to allow the entry of professional staff for inspection. Otherwise the inspection of the pipeline can only be carried out with specialized divers or ROV.

Structurally, the duct will have internal pressure equivalent to the total load of the reservoir and, externally variable depending on the loss of energy from the flow in the embankment plus the load of the embankment on the duct which will be variable depending on the height of the dam and will increase near the d/s control structure.

This unbalanced pressure condition can lead to leaks through the cracks in the duct/outlet/conduit that may form over time allowing water to flow from the duct to the embankment which, progressively, may get saturated to reach a state such that it begins to leak from the soil around the conduit. To reduce the possibility of leaks in these pipes, it is possible to internally protect the conduit with steel.

Unlike the previous case, the leaks that could occur in the conduit cannot be detected

unless there is a good instrumentation of the embankment around the conduct that alerts changes in the pore pressure of the body of the dam. For this reason, it is considered that the control downstream of the outlet work is less safe than the control upstream.

### **Mix Flow**

In those systems where the control structure is at an intermediate section between the inlet section and the terminal structure, it is usual to have pressure flow upstream of the control structure and free surface flow downstream of it.

In this case, the conduit can be inspected with some ease from the gate chamber to the downstream end; that is, the length with open channel flow. Upstream of the control structure can only be inspected in case the intake has emergency gate or stop logs or by specialized divers or with the help of ROV equipment. The gate chamber must be given access through a shaft/gallery specifically designed for this, either from the crest of the dam or from the downstream slope or the abutment of the dam.

The conduit will be in the upstream section will be subjected to pressure flow; that is, internal pressure will be equal to the hydraulic load of the reservoir relatively compensated with the external pressures that occur in the embankment corresponding to the phreatic line. There is a possibility of escape of water to the embankment dam.

Downstream of the control structure, the internal pressure will be virtually atmospheric while external pressures may be more important although they decrease as the terminal structure is approached. In this section, the generation of cracks or fissures in the joints or in the shaft of the conduit/duct may promote the entry of water into the conduit along with soil particles, which needs to be monitored periodically.

## Penstocks

Penstocks are a variation of the pressure ducts, which are used in Hydro-power development, through which the flow captured from the reservoir is transported to the powerhouse of a hydroelectric power plant.

The layout of these pipes is developed in such a way so that greatest energy efficiency (minimum hydraulic losses) can be achieved both in its layout and in its conduction: usually being steel pipes they are encased in concrete at all bends/curves to withstand the forces at such locations.

This type of conduit (penstock) may have a relatively short development, in the case of hydroelectric plants built at the foot of the dam or may have appreciable length depending on topography involved when water volumes are to be taken to another basin or river section to take advantage of the head involved.

The control structure is located in the vicinity of the turbines of the hydroelectric power station although in its route also it is possible that there are other control structures/devices that allow inspection of the penstock.

For details regarding penstocks specialist literature may be consulted.

### 4.3 Hydraulic Safety of Outlet Works

The Hydraulic Safety of the outlet works must be analyzed in two ways:

- The reliability of the control elements (Gates and valves) that allow the timely operation of the outlet works present in the reservoir.
- The preservation of the physical integrity of its components in order to avoid any direct and indirect deterioration of the hydraulic system/dam.

As already mentioned, a significant proportion of the failures in embankment dams are related to erosive processes initiated by the presence of conduits through the body of the dam.

Foster; 2000,commented *“About half of all piping failures and a quarter of accidents through the embankment are associated with the presence of conduits.”*

Zhang; 2009, has commented:

- a. The most common causes of embankment dam failures are overtopping and piping in the dam body or in the foundation.*
- b. For homogeneous earth fill dams and zoned earth fill dams, piping in the dam body/foundation is a dominant failure cause. Overtopping is also identified as an important failure cause.”*

In existing structures, verifying the hydraulic safety of the dam associated with the outlet works can be complex due to the number of processes involved, many of which are not easily detectable in their initial phases.

Fortunately, knowledge of historical failure processes has developed a series of design recommendations and construction specifications that greatly improve safety levels and reduce the potential for dam failure.

The safety of outlet works is to be ensured based on the safety assessment of each of its components when making routine inspections.

Further in control structures, (even in projects in which regular and efficient maintenance routines are developed), the safety levels, may be gradually reduced over time by the effects of aging on the Hydro-mechanical components besides the civil works.

Thus, the Hydraulic Safety of the outlet works should be considered as varying with time and, therefore, must be monitored continuously.

An important aspect in the outlet works lies in the dependence on the experience of personnel in charge of carrying out the operations of its valves and gates which is a key factor in its reliability in operation.

Another aspect that is becoming increasingly important is the effect of periodic inflow design flood review of dams, on outlets which are used for evacuating the floods.

A detailed description of the concerns from the point of view of hydraulic safety in respect of each component is given below.

### 4.3.1 Intake structure

The intake is the initial component of the outlet work and its function is to capture the water stored in the reservoir and to lead it to the conveyance structure with the minimum possible loss of energy and without clogging.

There is a wide variety of intake incidents; however, the most frequent incidents can be attributed to:

- Aging of the components of the outlet including trash racks, guides for the installation of stop logs, gates or valves, concrete and its aggregates and in general any component that may suffer wear and tear, physical deterioration etc.
- Sediment accumulation obstruction in the immediate vicinity of the intake structure
- Impact and / or obstruction on trash racks and mouth as a result of accumulation of floating debris or ice.
- Generation of vortices and incorporation of air into the intake works, reducing their hydraulic efficiency.
- Poor hydraulic designs that results in flow separation, negative pressures and, eventually, cavitation in control structures.

- Others: Structural problems associated with foundation, vibration, fatigue and seismic actions etc.

The various elements that make up the intake and can alter its hydraulic safety are analyzed below.

#### Trash racks

Although the topic related to waste, debris, ice, sediment and any other solids has already been dealt with in sufficient detail in Chapter 2, it is necessary to insist that in most cases the Inlet must have protection against entry of floating material and objectionable debris to the ducts and in some cases include tuff booms or other similar structure that minimizes the effect of floating debris on the structure.

#### REFER APPENDIX C

Retention by means of Floating Barriers (Tuff Booms) for effective design in the retention of debris, vegetation and ice

The design of the trash racks must be meticulously done in such a way that the accumulation of waste is not promoted, it is easy to maintain and structurally capable of withstanding the pressures that the accumulation of waste can generate. It is worth noting that the trash rack is clogged, the pressure differential between the reservoir and the duct increases and, at some point, the trash racks may fail.

The inspection of the trash racks must include:

- Level of obstruction of the bars and possibility of maintenance
- Physical conditions of the grid such as corrosion, evidence of impacts, deformations, state of the support guides, among others.
- Conditions of complementary devices, such as ice prevention systems, etc.

The bathymetric measurement of the bottom of the reservoir, especially in the vicinity of the intake, is vital to know the need or not to develop actions that prevent the advance of silt accumulation towards the intake.

Numerous reservoirs include sediment flushing in their operational routine in order to minimize the risk of obstruction of the orifice type of spillway or the breast wall type spillway of the lower intakes of the reservoir.

### Inlet

As with the trashracks, the inspection of the bell mouth is complex except for the sluices and other discharge carriers located in sectors of the dam close to its normal water level (FRL). It is also feasible to observe the conditions of the openings in multilevel intakes especially those located at a low depth.

The most important consideration in the Inlet design at the inlet section is that the flow lines must ideally flow smoothly in a gradual and progressive manner without any separation from the inlet profile or generation of vortices, flow separations or stagnation points.

The most visible effect of a poorly designed inlet is observed with the incorporation of air into the flow, the generation of vortex around the intake and, eventually, the deterioration of the inlet surface due to these disturbances in the flow coupled with entry of suspended sediment that impacts the structure.

Adaptations/Modifications of the inlets in built dams are normally very difficult and feasible only if it is possible to reduce the level of the reservoir for its execution.

The usual shape of the mouths in rectangular and circular conduits is elliptical, and its design can be defined with the equations presented below although in hydroelectric uses it is common to develop

physical models that reproduce the inlet conditions and develop geometries that prevent the formation of vortices, effect a gradual transition of the velocity and minimize the drag of solids towards the conduit.

The Indian Standard: 11485-1995, Criteria for Hydraulic Design of Sluices in Concrete and Masonry Dams & IS 9761 Hydropower intakes – Criteria for Hydraulic Design and the Design Manuals of the Army Corps of Engineers adopt elliptical shapes that follow equations depending on the cross-section of the conduit. For equations of the Inlet the above Indian Standards as well as a lot of other technical literature of USBR/USACE can be referred to

Another aspect of importance in the entrance section is the need to prevent the formation of vortices and, thus, the incorporation of air into the conduction structure. (See Figure 4-34 to Figure 4-37).

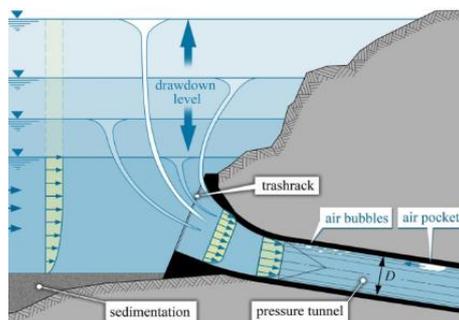


Figure 4-34: Schematic illustration of air entraining intake vortices. (Möller, 2013)

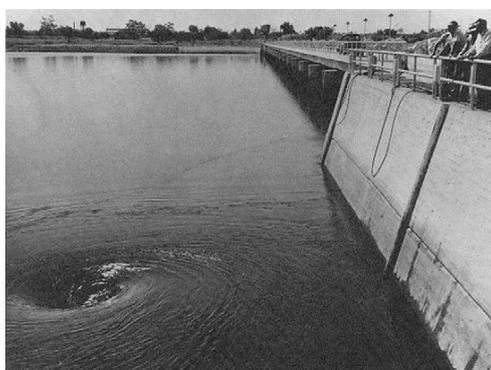


Figure 4-35: Vortex at intake of hydropower plant (Möller, 2013)



Figure 4-36: Vortex at diversion tunnel Lake Arapuni, NZ (Möller, 2013)



Figure 4-37: Guri Dam. Vortex at intake hydropower plant (EDELCA, 2001)

According to Möller (2013), the vortex formation process is still misunderstood, there is no criteria to define a critical submergence, nor can the amount of air that can be incorporated into a conduit with the development of a vortex be estimated with some precision; however, it is concluded that the formation of the vortices can be erratic with either a low incorporation of air into the system or a stable and a constant incorporation of air. The smaller the submergence of the structure, the greater the possibility of generating vortices and these will be more stable.

In general (Knauss, 1987) suggests adopting the result of the following dimensionless equation as a minimum submergence (S):

$$\frac{S}{D} = 1 + 2.3Fr$$

where,

S= Minimum Submergence  
 D= Vertical dimension or diameter  
 Fr= Froude number of the sluice for the design condition

### 4.3.2 Control structure

The hydraulic safety of the control structures must be analyzed from several perspectives since their operation can be compromised by operational errors (human), waste, ice, corrosion, aging, structural, mechanical, electrical problems and, of course, hydraulic deficiencies.

This Manual emphasizes hydraulic problems that could arise from a poor design that may lead to possible failures of control devices, deterioration of surfaces or transient flows that damage the structure in general.

Failures of the control devices do not normally compromise the safety of the dam but could prevent the realization of some of its functions and, thereby, reduce the life of the dam or it may reduce the benefits accruing from the dam. However inoperative spillway gates affect dam safety.

In sluices and other structures that are used to control the reservoir levels, as well as in other bottom outlet structures that are used for flood and sediment evacuation, the failure of some of the control equipment can increase the reservoir levels and, with it, damage other structures or cause the overflow of the dam and with it the general collapse.

The structural, mechanical and electrical problems encountered in valves and gates are covered in the “Guidelines for Preparing Operation and Maintenance Manual for Dams” CWC, 2018.

The hydraulic actions that compromise the safety of these elements are described below.

#### Cavitation

The main hydraulic problem that should be avoided in the control structures is cavitation and, especially, in those gates or valves that are required to operate partially open.

Modifications of the stream lines, abrupt changes in the flow velocity, abrupt modifications of the contours/geometry and anomalous flow interaction with the boundaries result in generation of vortices, secondary flows, stagnation points among others, precipitate the formation of areas where the pressure variations is very significant and the appearance of vapor bubbles is likely.

Within the control structures, the gates are the ones that are most affected by cavitation. When it is desired to control the flow with valves these are usually located near or at the lower end of the conduction structure where the supply of air from downstream is guaranteed.

The effect generated by cavitation in the control structure is visible both in the control device and on the surface of the conduit where it is installed (See Figure 4-38 to 4-42)

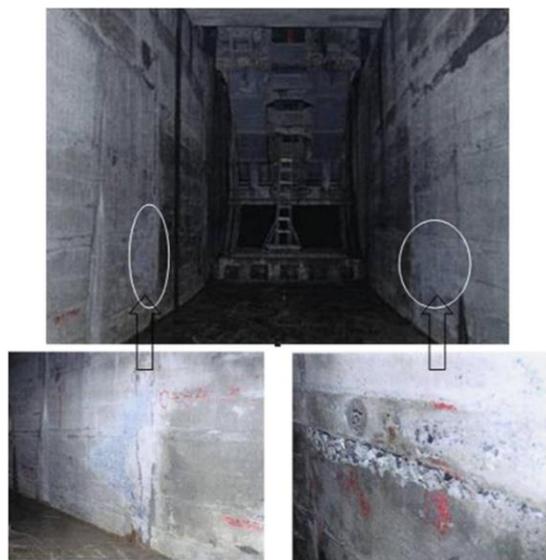


Figure 4-38: Cavitation damage zones on the side walls in flood-discharge tunnel. (Shuai, 2016)

To avoid cavitation immediately after the gates and valves, it is necessary to install air vents that inject enough air to prevent the generation of areas with pressures below the water vapor pressure.

The implantation of the control devices in many occasions requires modifying the cross section of the sluice. Usually, upstream of the valve or gate it is necessary to provide a contraction in the section and to subsequently provide an expansion on the downstream.

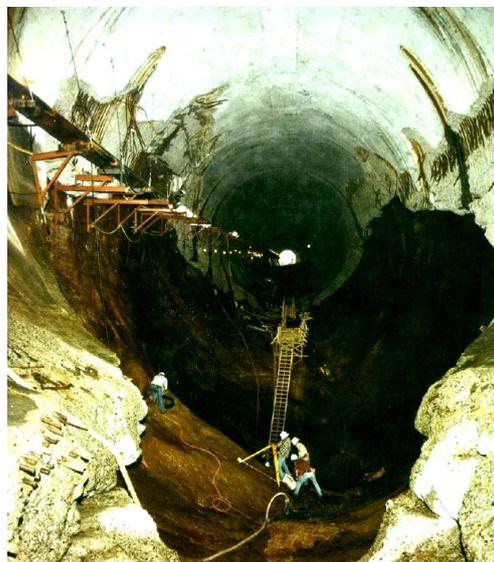


Figure 4-39: Glen Canyon spillway tunnel failure. Air slot was later constructed to provide air supply & avoid cavitation

Both transitions, if performed abruptly, can cause the separation of the flow and the occurrence of the cavitation phenomena.

Ventilation conduits/Air vents are not always capable of supplying air in sufficient quantity to prevent the occurrence of cavitation. In some cases, the cross section of the air conduit, the distance to which fresh air is captured or the partial or total obstruction of its inlet prevents the delivery of sufficient air volume to the water flow and, as a consequence, cavitation

Associated with cavitation, the high velocities that are generated in the control section and the variation of dynamic and hydrostatic forces, the control devices are exposed to vibrations that in some cases lead to deterioration of the structural elements, fatigue, loss of alignment, blockage of parts and, in some cases, failure of the structural element.

The causes that can compromise the reliability of the control devices can be summarized as:

- Aging: This alters the efficiency of the equipment and generates weakness points that may result in leaks, operational problems and reduction of thickness of various members.
- Lack of maintenance: This accelerates processes of corrosion and blocking of drainage holes in various parts of the gates due to accumulation of waste material.

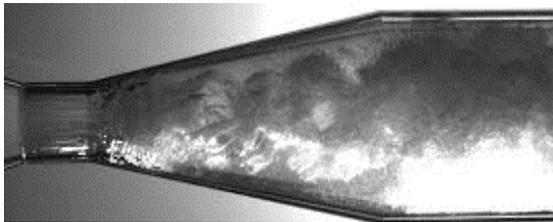


Figure 4-40: Cavitation in an abrupt expansion  $\sigma=0.33$  (Kosak, 2016)

- Alterations in control mechanics due to impact, friction, ice, among others.
- Operational impediments due to the absence of electricity supply.
- Human errors during operational processes or in decision making altering the established operating rules.
- Affectation of mechanisms in the devices due to lack of use.
- Structural deformations due to settlement, foundation failure or construction failure.
- Vandalism.



Figure 4-41: Globe Valve Cavitation damage (General Technical Knowledge; 2017)

### 4.3.3 Conveyance structure

The conveyance structures are planned to pass the required flow and its shape, location, levels and dimensions are established based on the hydraulic design.

The hydraulic safety of these structures must be seen from two (2) different angles. i.e. the safety criteria associated with the structure itself and the general safety of the dam as a result of the presence of the conduits in the body of the dam.

In the first case, hydraulic safety refers to the guarantee of the timely and efficient functioning of the work for the purpose for which it has been designed, while in the second case we must establish how the overall integrity of the dam can be affected by problems associated with the conduit and its contact with the dam body.

As already mentioned, a large number of problems associated with failures in embankment dams originate at the contact of the conveyance structure with the dam embankment; these problems may get initiated either from inside the outlet work or from outside i.e. from the embankment.

Further in case of conduction structures passing through embankment dams, the general safety of the dam can be compromised if there is a failure of the conduit or if there is leakage through it or if at the interface the fill is not well compacted.

Conveyance structures such as tunnels in rock or sluices in concrete/masonry dams do not usually compromise the integral safety of the dam.

### Hydraulic Safety

The hydraulic problems associated with conduits/outlets can be summarized :

- Cavitation damage: Associated with problems in the geometry of the conveyance structure, irregularities in the

surface of the concrete or insufficient air



Figure 4-42: Cavitation in Tunnel Spillway of Glen Canyon Dam (USBR, 1983)

injection in low pressure areas.

- Abrasion damage: Generated by the presence of high concentrations of solids transported by water and, especially, in the bottom outlets, sluices, and sediment discharges.

### **Cavitation**

Cavitation processes are developed as a result of the conjunction of two aspects that are basically summarized as: high velocities and local disturbances of current water passage:

The hydraulic design of the conduction structure must be carried out in great detail to prevent generation of flow separation zones that lower the pressure below the vapor pressure of the fluid and, thus, the formation of bubbles and implosion posterior of these inside the conveyance structure.

The area's most vulnerable to the occurrence of cavitation in outlets are those where poorly designed short transitions are provided or where control equipment (gates, guide valves or others) are located.

Further locations where there are changes in horizontal and vertical alignments are also prone to separation of flow.

The deterioration processes of the control structure due to cavitation are usually progressive and incremental; that is, once the cavitation process begins, the effect of pitting on the surface of the conveyance structure generates new irregularities of the contour that in turn generate greater areas of separation and the occurrence of larger and larger areas of bubble generation.

As can be seen in Figure 4-42, the cavitation processes gradually destroy the conduit structures, causing them to collapse.

In concrete and masonry dams it is possible to correct these failures in most cases; However, if this fault occurs in the body of an embankment dam, the conduit failure is the beginning of an internal erosion process of the embankment that in a short time can cause its total collapse.

As already mentioned, changes in the geometry of the constructed pipelines is practically impossible or would result in a decrease in their hydraulic conduction capacity, so improvement measures must be associated with detailed inspections; repair of damaged areas or, in some cases, the installation of steel shielding in areas where fluctuations in flow pressure could reach the vapor pressure of water.

The incorporation of air in areas where low pressures are expected is one of the measures that can avoid the problems associated with cavitation, so during the design phase it is common to incorporate air vents/ air entrainment measures to maintain atmospheric pressures in sections where important flow modifications occur.

On some occasions, the air incorporation structure does not fulfill its function properly either due to a poor design, or difficulties in the entry of air into the

structure due to blockages or by the way in which the air is distributed within the body of water. In such circumstances even if the facilities that could prevent cavitation processes are present, the deterioration of the bottom and walls of the conduits may happen as a result of the impact forces that occur when imploding water vapor bubbles.

The conduits where the flow, downstream of the control structure, is kept at free surface, are usually very sensitive as a result of the high velocity and low pressures that exists therein and require a suitable air vent design.

As mentioned in Chapter 3, the cavitation index ( $\sigma$ ) provides idea of the potential cavitation due to flow in the conduits. Flow velocity above 20 m/s and cavitation index  $\sigma$  less than 0.30 are indicators of a high probability of cavitation in the conduits. In this connection IS 12804 – Criteria for estimation of aeration demand for spillways and outlet structures may also be referred to.

The cavitation process can also begin in those spaces of the bottom surface and sluice walls where there are irregularities in its surface.

These irregularities can be due to multiple factors: processes of concrete degradation, aging, abrasion, bad construction practices or even deformations in the conduit as a result of settlement etc.

As previously mentioned, this process, if not monitored by regular inspections, is progressive and will increase over time and generate greater cavities. In outlet works, the frequency of use can be very high and the operating times are long, so once this process has started, the physical integrity of the conveyance structure can deteriorate very fast.

## Abrasion

It refers to the erosion process that occurs in the conduction structures due to the effect of flowing water at high velocity, carrying large quantities of sediments and other materials; many all of them with great abrasive potential (See Figure 4-43).

The abrasion process can occur under any flow condition although it is enhanced when sediment concentration and velocity are high. Obviously in bottom outlets and other intakes that are designed for the evacuation of sediments, large erosive processes should be expected as well as in sluices or any other conduit that could carry suspended sediment particles of any dimension.

In addition to the erosive processes existing in the conveyance structure, impacts of larger solids can alter the surface of the conduits and, in some cases, affect the guides of the gates or other elements embedded in the conduits.



Figure 4-43: Bottom outlet removing sediments. Xiaolangdi Reservoir, China ([www.news.cn](http://www.news.cn))

Abrasive processes tend to be more significant in those conveyance structures through which the flows are very frequent and also in old conduits where materials have aged and where more aggregates and rocks accumulate as is usually the case in respect of dams located at the foot of the great mountains.

It is commonly accepted that any flow that exceeds a velocity of 12 m/s (40ft./s) has a high abrasive potential and this velocity is widely exceeded at the bottom or

intermediate outlet of dams with a drop of more than 20 meters.

Abrasive processes usually begin with the loss of surface concrete until structural reinforcement is reached and the process continues until the section is structurally weakened.

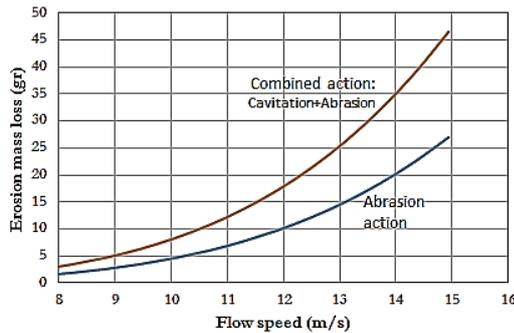


Figure 4-44: Relationship between erosion and flow velocity (Wang, 2019)

In some cases, the loss of surface material due to the cavitation gets added to the loss due to abrasion process as a result of the separation induced by the irregular surface generated by abrasion. In these cases, the degradation of the material multiplies and the damaging effect on the structure accelerates over time.

Figure 4-44 shows a graph developed by representing the results obtained by Wang (2019) in his study of the erosion mechanisms on the surface of the concrete due to the effect of abrasion and cavitation. Also see Figure 4-45.

As expressed therein, abrasion occurs from early velocities but has an exponential



Figure 4-45: Concrete surface after abrasion test (Wang, 2019)

behavior as the velocities and, with it, the power of erosion due to the flow increases. If this condition includes a degradation process as a result of the occurrence of cavitation, the damage will be even greater and progressive with the consequent probability of structural failure.

Regular inspection of all the outlets/sluices is most important and repairs of damages may be carried out in initial stages.

### Conduit induced failures

Figure 4-46 shows four (4) failure modes associated with the presence of the conduits/outlets in an Embankment dam. They are:

- Internal erosion of the embankment with seepage water/soil particles entering in the conduit
- Conduit leaks cause piping in the embankment.
- Piping along the dam-conduit contact

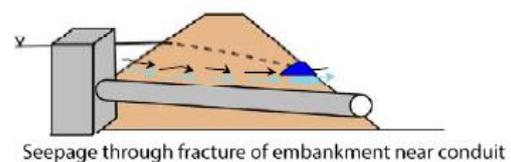
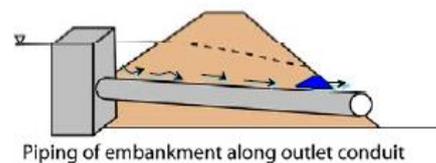
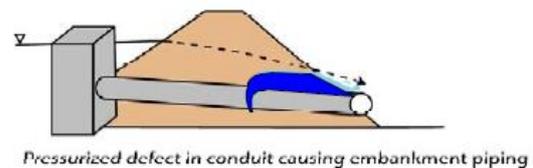
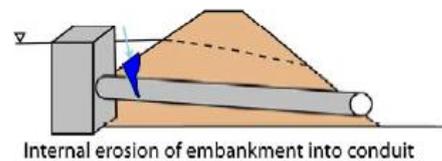


Figure 4-46: Failure Modes for Conduits in Embankment Dams

and

- Seepage through cracks near the conduit.

In the first case and with the conduit working at free flow, any cracking in the conduit as a result of foundation problems, construction problems, deformations of the conduit due to pressure imbalance could lead to seepage of water from the embankment dam into the conveyance structure.

Seepage of water may cause soil material to be carried away in a process that, although slow, is continuous due to the difference in pressure between the water contained in the soil around the conduit and the internal pressure inside the conduit.

Depending on the location where such seepage occurs and the flow of the same, the material dragging process may compromise larger areas until slides in the upstream or downstream slopes of the dam result.

In the second case and unlike the previous one, the control structure must be located somewhere in the dam body and, as a consequence, the conduit flows full and pressurized from the upstream up to the control structure.

Under these circumstances, if any displacement of the joints occurs, erosive or cavitation processes will cause seepage/leakage from the conduit (high pressure) to the body of the dam (low pressure); these leaks will follow the contact of the dam-conduit towards the downstream face of the dam and, once it proceeds to drag soil particles, a piping process may get initiated.

In some cases, such seepage/leakage occurs at the lower level of the conduit and, in that case, in addition to the above-mentioned process, saturation of the foundation soil, loss of bearing capacity, differential settlement in the structure can occur. Increase in cracks that extend leaks and

further promote erosion, may ultimately cause the embankment to fail.

The location of the leak that initiates the process is of vital importance since if it occurs in areas near the downstream it is possible that its effect is local and its repair feasible without greatly modifying the safety levels in general. However, if the leak occurs in areas near the axis of the dam and near the upstream face, the consequences of this type of failure can be extensive and it may cause the general collapse of the structure.

Normally the seepage or leakage processes from the conduit begin through cracks generated in the concrete/masonry or in joints where the measures taken to prevent the passage of water have deteriorated due to aging or degradation effects that could exist internally on account of abrasion or local cavitation.

The third and fourth cases are associated more with construction problems than with problems related to leaks from or to the conduit. In both cases the conduit is the reason why a preferred waterway or a weak sector is generated within the dam embankment whereby the water from the conduit finds a shortcut to join the water seeping from the reservoir and flows towards the downstream slope. In the third case, the preferred route that the water finds is the contact surface between the conveyance structure and the dam embankment. Although collars and other structures that increase the length of seepage at this contact are usually incorporated, the presence of very old dams could lack these elements and therefore decrease the hydraulic safety of this work.

Figure 4-47, on the other hand, shows the failure of a dam even when the conveyance structure had collars for the prevention of such problems; nevertheless, later studies revealed that the use of collars makes compaction work difficult, generates problems of differential settlement and, in

many cases, it seems that it may have favored the formation of a preferred waterway. Updated designs do not prefer the construction of collars and the side walls of the outlet are kept sloping for better contact/compaction of earth fill at the contact or a diaphragm filter that captures the waters that potentially seep/drain between the conduit and the embankment and the dam.



Figure 4-47: Internal erosion around conduit in embankment dam

In the event that a free water path is developed as described, the loss of soil as a result of the dragging of material by the water stream will be increased from the leak site to the discharge site located on the slope downstream of the dam.

The fourth case is usually called hydraulic fracture that can occur when the seepage pressure inside the embankment exceeds the contact pressure between two layers of soil in the same plane of the dam embankment.

Once hydraulic fracturing begins, a gap/crack is created in the dam that acts as a conduit within the body of the dam for seepage/leakage to flow in progressively increasing amounts and at a faster velocity and with greater capacity to mobilize migration of soil particles. Hydraulic fracturing is more likely in zoned dams where there is a possibility of differential settlement between the different materials that make it up.

While cracks in outlets/sluices can occur in both embankment and masonry / concrete dams, the cracks in sluices located in

masonry/concrete dams can be treated more easily and the failure of the dam is less likely.

The tunnels excavated in the abutment of the dam or any another location near it do not generate erosive problems that may compromise the embankment; however, abrasion and cavitation processes that compromise their physical integrity, can occur.

Regular inspections and supervision of the dam is the activity that can assist to avoid such type of failures. Modifications/changes in the pore pressures detected by instrumentation of the dams can also provide alerts on any likely incipient seepage/leakage processes and, once they are detected, necessary rehabilitation measures to prevent them from further aggravating the situation are required to be taken although the repair or rehabilitation process may be quite complex.

It is worth noting that those conveyance structures in which the control structure is located upstream, or near the upstream face of the embankment dam, are easier to inspect and in case of any damages it is feasible to carry out repairs by lowering the gate and isolating the conveyance structure, without affecting the integrity of the dam. However, in those conduits where the control structure is not near the upstream face but somewhere inside in the dam body, the conduit cannot be inspected in its full length internally and only a follow-up of the instrumentation in the body of the dam or a meticulous inspection of the downstream slope can provide a warning of the processes of incipient internal erosion..

#### **4.4 Rehabilitation Measures**

In this subchapter, a brief description of rehabilitation measures in existing dams in order to improve the performance of the outlet work.

Technical Manual: “Conduits through Embankment Dams Best Practices for

Design, Construction, Problem Identification and Evaluation, Inspection, Maintenance, Renovation, and Repair”(FEMA, 2005) is a good reference.

The rehabilitation measures are described considering the components of the outlet works in the same order as described earlier.

Some of the measures described, as well as the recommended materials for their execution, can be further supplemented from the “Manual for Rehabilitation of Large Dams” issued by CWC in 2018.

#### **4.4.1 Intake structure**

The hydraulic problems associated with the intake structure are basically related to:

- Aging of the components.
- Obstruction of the intake structure
- Impact by floating debris or ice.
- Vortex formation
- Poor hydraulic design

##### **Aging**

This is a well-known situation, for which are protocol of action.

Obviously, all electro-mechanical components, will have a useful life less than that of the dam and therefore, periodic removal and replacement is necessary.

Normally every intake has devices like gates/valves that require removal and replacement of the components with time. In some cases, these activities can be carried out without altering the normal functioning of the reservoir; in other cases the level of the reservoir is required to be lowered and, in some other cases, divers or other specialized equipment are utilized for execution.

The measures to be taken against aging are basically: the maintenance and preventive

care of all components during their operational period and, after that, their replacement taking into account the original design specifications with the technological improvements that may have been developed in the recent times. It is worth mentioning that any change in the original design conditions must be analyzed so that at no time are operational changes generated that may alter the work's function.

Those components that require lowering the levels of the reservoir (in absence of stop logs or emergency gate) to proceed for its replacement normally generate greater reluctance on the part of the dam owners to undertake the work since this may result in the partial or total cessation of the benefits accruing from the dam with effects on the concerned population which benefits from the same and associated economic losses.

It is for this reason that in most cases new alternative options are explored that do not involve the lowering of reservoir levels or, in some cases, in delays in replacing the component. This situation in turn can involve two scenarios in the security of the system. The first is associated to the component since the delay in the replacement of the component can compromise the general safety of the work and can lead to its collapse and the second, is associated with increase in the levels of insecurity of the technicians/operators in charge of operating them.

##### **Intake Obstruction**

The rehabilitation measures associated with the obstruction of the components of intake by sediments, floating residues and ice are discussed in good detail in Chapter 2 of this manual and are aimed at minimizing the effects of these obstructions in reducing the effective area of the intake and at minimizing the impact of the floating elements with the trash racks and entrance inlet in the intakes.

The incorporation of air bubbles in the inlets in order to avoid the formation of ice in its components is one of the rehabilitation measures of the intake.

### **Vortex Formation**

An exhaustive study is required that normally includes study of the intake in a physical model in order to analyze the reasons why it occurs and the measures that must be taken in this regard to prevent its formation. A vortex formation at the intake implies increase in energy losses.

The rehabilitation measures associated can vary and all of them are required to be studied in the physical hydraulic model study mentioned above. Some of the rehabilitation measures adopted in this regard can be: install anti-vortex structures, improve the shape of the entrance mouth, eliminate guides and other supports that promote secondary flows, eliminate circulation flows, etc.

### **Poor hydraulic design**

It can reduce the efficiency of the intake by increasing losses or generate operational problems in the intake or in the hydraulic performance in the inlet that induces vibrations, pulsations in the flow or cavitation.

The rehabilitation measures to be carried out in these cases may include replacement of the control elements that favor the prevailing operating conditions, incorporation of air-vents that prevent the occurrence of cavitation processes, improvements in the transitions of entry to the duct, etc.

#### **4.4.2 Control structure**

In the case of control structures, without taking into account the structural or electro-mechanical problems which are discussed in other guide lines developed by CWC, the hydraulic problem is reduced to 2 basic aspects: Aging and Cavitation.

Other measures associated with improvements in the Hydraulic Safety of control structures have to do with their reliability and, the personnel in charge of their operation.

- For this reason and by slightly broadening the hydraulic nature of the document, the following measures are considered important: Training of personnel in the operation rules of control devices and routine maintenance.
- Periodic testing of all the devices that allows to verify its operational status even in those whose actuation is eventual or is associated to particular situations.
- Regular Inspection of all the equipment, components and pipelines that constitute the intake works.

### **Aging**

This aspect has been discussed earlier under intake structure. Cavitation – Air injection

The causes due to which cavitation occurs can be varied but in all of them it has been shown that air injection in the area where it is possible for cavitation to develop is the best way to prevent its formation.

On certain occasions, air-vents designed to maintain adequate conditions in critical areas exist and, nevertheless, the phenomenon occurs either because of the inability to supply enough air or because the design is not suitable for the specific operating condition.

In any case (absence of air-vent or malfunction of it) it is necessary to define the amount of air to be incorporated and, for this, IS 12804-1989 standard “Criteria for estimation of Aeration Demand for Spillway and Outlet Structures ”can be referred to.

### **Cavitation – Rough surface**

On the other hand, the cavitation process is progressive and feeds on the surfaces of the duct that are damaged by this or some other effect.

In this sense, the repair of the affected surfaces is a complex task that must include use of specific high-strength materials whose effectiveness has been proven in other projects.

Some of the materials used for repairs include steel lining, high strength concrete reinforced with steel or polypropylene fibers, epoxy resins, etc. Details on these and other materials and their application in this type of rehabilitation work can be found in the “Manual for Rehabilitation of Large Dams” (CWC, 2020).

It is important to emphasize that the material in the repair constitutes a temporary improvement of the surface while the ventilation can be a definitive solution of the problems and, therefore, on surfaces already affected by cavitation both works must be carried out: prevent their formation and repair the surface.

In any control work where there have been damages related to cavitation, it is essential to redouble vigilance in relation to the efficacy of the rehabilitation measures adopted. For this reason, it is essential to carry out inspections frequently in order to observe the efficiency or not of the work carried out and proceed accordingly.

#### **4.4.3 Conveyance structure**

The, rehabilitation measures are aimed at:

- Prevent the deterioration of the outlet work due to the effect of cavitation and / or abrasion
- Prevent the failure of embankment dams as a consequence of problems associated

with the presence of the duct in the body of the dam.

### **Conduit rehabilitation**

The ducts, especially those carrying high concentration of sediment may get damaged due to abrasive processes, cavitation, as well as chemical attack on their basic components.

A deteriorated conduit that is suspected of leading to an internal erosion process in an embankment dam must definitely be rehabilitated. The logical answer, as mentioned in valves and gates seems to be removal and replacement with a new one; however, in dams built and operating this solution, could only be considered for works that are at higher levels where these activities can be carried out without implying the interruption of the dam's service.

In any other condition and, especially in outlets located close to lowest river bed, the replacement of a conduit would imply the emptying of the reservoir, excavation, extraction of the conduit, the placement of the new conduit and the restoration of the embankment dam. The magnitude of these works, the time required for their execution and the impact generated on the population benefitted makes this option very difficult for adoption.

Thus, the rehabilitation of the conduits refers to the execution of works without implying the interruption of the service or the emptying of the reservoir trying to restore the operating conditions of the original conduits with new materials that assign greater security and extend its useful life.

In general, three methods of conduit rehabilitation are accepted:

#### **Sliplining** (See Figure 4-48 and Figure 4-49)

It refers to the installation of a smaller diameter pipe inside the original duct, leaving a space between the two structures

that is usually filled by injection with mortar or grouting.

Additional activities that are usually necessary for this type of work include the preparation of the inlet and discharge works and, normally, the incorporation of a filter diaphragm in the section downstream of the dam.

The reasons that can justify this type of work compared to conduit replacement include:



Figure 4-48: Replacement of Pipe in Dam (KRG Utility)

- Maintenance of the reservoir level: During the execution of the work it is not necessary to modify the operating levels of the reservoir, so it is possible to keep the system operational for the function for which it was designed.
- Time: Normally this type of work is done in a short time, so the impact on the population is usually less.
- Complementary work required: Excavation and filling works are minimized in the dam, reducing in most cases to small areas near the ends of the conduit.
- Costs: Compared to the costs associated with the removal and replacement of the conduit, the costs associated with sliplining are much lower.

As in any work, in this type of process the disadvantages that must be weighed when making decisions must be taken into account. Some of them are described below:



Figure 4-49: Annular space grouting Replace pipe in dam by a smaller diameter pipe (Mays construction)

- In order to carry out this process, a relatively constant section must be provided along the entire conveyance structures and with a straight alignment or with few changes in direction. If deformations occur as a result of local structure failures or sudden or continuous changes in alignment, it will not be feasible to perform the sliplining.
- The development of this activity requires specialized personnel and equipment both for the sliding of the new conduit and for the sealing of the space between the new and old conveyance structures.
- It is not always possible to carry out the replacement work with the reservoir completely full and, therefore, it may be necessary to lower the reservoir even for a very short period. The answer on the possibility or not of carrying out this emptying will depend on the possibility or not of preventing the entry of water from the intake
- Before proceeding with the replacement work, it is advisable to analyze the general condition of the dam and the conduit and in particular they should verify:
  - The poor physical condition of the original conduit that may cause it to crumble while the new conduit is slipping and thus generating a primary gap in the body of the dam that degenerates into a general failure of the work.

- Seepage/Leakage from or into the old conduit presumes a condition of saturation of the soils around the conduit. The incorporation of the new structure will entail a modification of the internal pore pressure in the embankment and possible instability of the embankment dam.
- The location of the leaks in the conduit can be worrisome or complicated to handle. In principle, it will be assumed that seepage leaks downstream of the dam require more attention.
- Existence of voids behind the conduit constitutes a local weakness in the body of the dam that must be corrected. The filling of these voids with mortars is a good option although it could increase the costs associated with the work

Once the sliding process and the sealing of all the gaps between both conduits have been completed, a system with a long service life must be guaranteed.

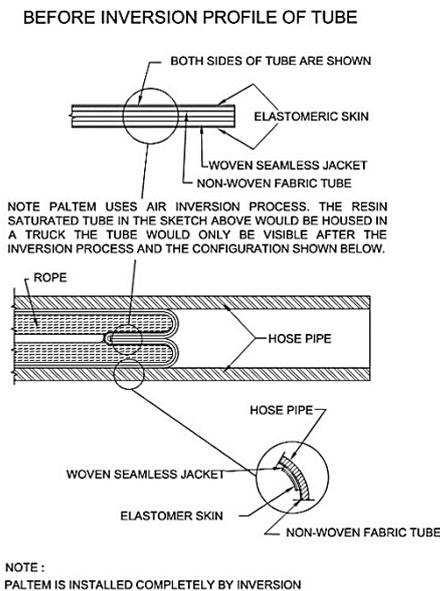


Figure 4-50: Typical CIPP installation by air inversion (Patem HL) (USACE 1995)

The new structure must be able to resist the pressures (internal and external) that the original conduit resisted and, therefore, constitutes a good option when compared

with the total removal and substitution of the work.

### Cured-in-place pipe (CIPP)

The CIPP is a saturated liquid material of thermosetting resin that is usually used in those outlets where access is limited and it is not possible to work internally. (theoretically this measure of rehabilitation is not limited to small diameters).

This system can be used in rectangular sections and eliminates leaks to and from the pipeline as well as corrects defects in the surface of the conduit and reduces the effects of corrosion in metal pipes or metal reinforcement. It reduces the coefficient of friction in the pipes and, therefore, they improve the conduction capacity although the installation of this system reduces the internal diameter of the conduit, which is why the possibility or not of developing this technique should be analyzed in detail.

Above Figure 4-50, taken from the USACE Guidelines for trenchless technology, shows the installation techniques of the CIPP system by air inversion and can also be installed by hydrostatic inversion and mechanical means(See also Figure 4.51).

Once the material is inserted, it will take the form of the original conduit with a slightly smaller dimension depending on the thickness of the material placed. Subsequently, the material is quadrated by applying hot water or steam.



Figure 4-51: CIPP installation at Upper Taylor Dam, Powell County, Montana (Fischer 2009)

Before proceeding with the installation of these materials it is advisable to ensure that:

- The length to be covered is less than the maximum allowed by the technology used. Usually the range is between 300 and 900 meters in length.
- The pipe does not show deformations or severe damages that prevent the installation of the equipment. It is also not convenient with abrupt changes of direction.
- The capacity of the pipe, once the system is installed with the recommended thicknesses, is able to pass the water flows with the same magnitude and conditions that the project requires.
- It is, from the economic point of view, the best option for this outlet work.
- It is possible to temporarily suspend the flow through this conduit.

The insertion of CIPP into pipes has the following comparative advantages:

- It is a continuous element that ensures the reduction / elimination of leaks in the conduit.
- The inner surface is smooth and normally compensates for the reduction in effective area within the sluice with less opposition to rough motion
- Grouting injection is not required to fill existing holes
- It can be used in non-circular conduits and in pipe sections with slight changes in direction.
- The costs associated with its installation are competitive even in large-scale project
- Relatively fast execution times.

On the other hand, among the disadvantages that can be noticed are:

- The costs of small project installation can be high compared to other techniques.
- Specialized personnel and equipment are required for installation.
- The use of the conduit must be limited by the period of execution of the works and that corresponding to the curing of the materials.
- If the conduit to be rehabilitated has significant deformations, the material will adopt the current form, in no case is the original form restored.

### Spray lining (Figure 4-52)

It is a method applied for several decades which consists in the application by centrifugation and spray of a mixture of mortar, epoxy or resin against the inside of the original sluice beds.

During the application of the coating, there are usually rotating elements that standardize the application and soften the surface finish.

This type of coating is suitable for conduits where there are abrasion or corrosion processes and it is desired to prevent them from progressing but it is not functional in conduits which are structurally unsafe since it is not suitable for resisting any pressures.



Figure 4-52: Spray lining material for conduit (D.A.V. 2020)

Used mostly in low-height dams and should be inspected frequently since coatings do not usually have a long service life.

## Filter Diaphragms

Most of the internal erosion failures in embankment dams occur adjacent to the conduits that cross these embankments and their causes, as already mentioned, can be very varied but in all cases there is a common element: seepage/leakage water flowing through the soil surrounding the conduit(Figure 4-53).

In most cases, these flow concentrations are associated with construction problems due to difficulty in access of the equipment in areas close to the outlet/conduit or due to the development of fragile areas around the outlet/conduit as a result of leaks, compaction with reduced energy and hydraulic fracture.

In general, the solutions that have been used to combat this problem are:

- Construction of anti-filtration collars
- Installation of Filter Diaphragms.

Anti-filtration collars were the most used solution between the 60s and 80s (USDA, 2007); However, experience of some failures suggest that this solution did not generate improvements in the designs but on the contrary developed an area of high fragility in the body of the dam.



Figure 4-53: Typical configuration for a filter diaphragm. (FEMA, 2005)

Theoretically anti-filtration collars had 2 virtues. The first is that they prevented direct contact between the embankment and the conduit and, the second, any seepage/ leakage from the conduit, increased its travel and therefore reduced its hydraulic gradient and thus its ability to transport sediments.

Dams built with anti-filtration collars failed even during the first filling (Anita Dam, 1997) and, although the design of the collars was correct, the fractures/cracks were generated by problems associated with the soils located around the conveyance structures.

As described in the National Engineering Handbook, Part 633, Chapter 26 “Gradations Design of Sand and Gravel Filters”, the research carried out at the Soil Mechanics Laboratory in Lincoln Nebraska presents criteria for determining the grain size distribution (gradation) of sand and gravel filters needed to prevent internal erosion or piping of soil in embankments or foundations of hydraulic structures, developing an element that some call the “No erosion Filter”.

This filter demonstrated that it was efficient in intercepting the flow and sealing the crack or water path and, thus, avoiding the possibility of erosion or piping.

Considering these two facts *viz.* dam failures with anti-filtration collars due to internal erosion and development of a No erosion filter, the recommendations have been modified and anti-filtration collars have been eliminated and the construction of diaphragm filters is encouraged.

A Filter Diaphragm is an element built around a conduit filled with sand and gravel with a design that prevents the transfer of particles downstream of the diaphragm (See Figure 4-54 and Figure 4-55).

As can be seen in Figure 4-54, the filter diaphragm is constructed around the duct in the downstream (d/s) section of the dam and up to a level equal to or greater than the normal water level of the reservoir. A draining filter is constructed under the conduit and downstream of the filter, terminating at the foot of the downstream slope of the embankment.

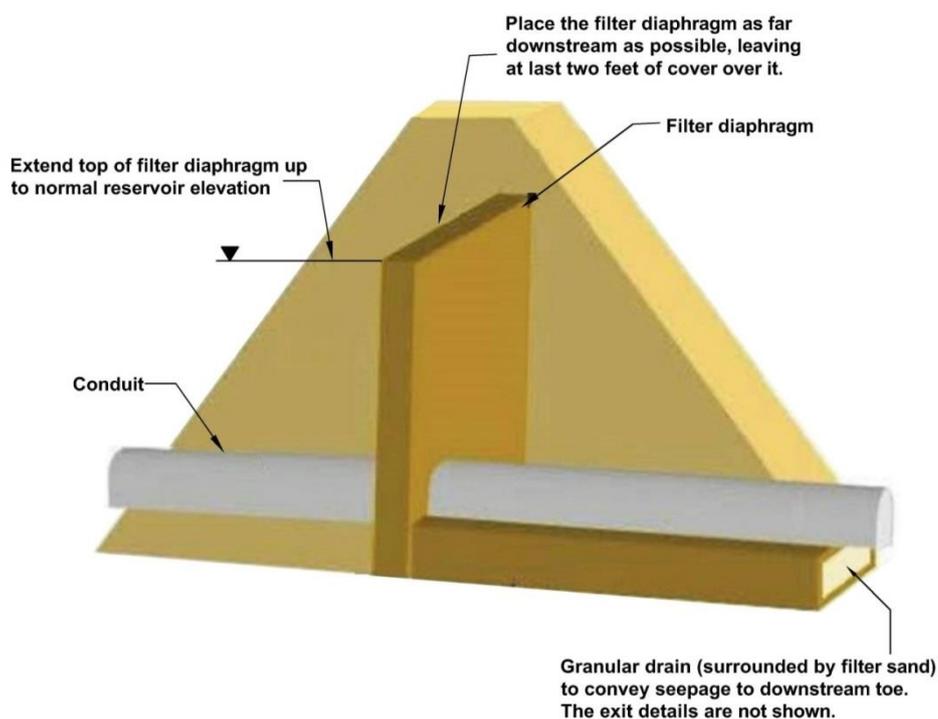


Figure 4-54: Typical configuration for a filter diaphragm. (FEMA, 2005)

The diaphragms have the advantage that they can be constructed in a new dam and at the same time they can be incorporated into a dam already built as an element of protection against possible present or future leaks. In some zoned embankment dams, there are chimney filters whose function is very similar to that of the diaphragm and therefore its construction in such sections would be redundant. The filter is designed to stop fine particles that can be driven by the flow.

These fines retained by the filter will generate a layer of fine materials that will progressively decrease the permeability of the layer, filtration and fill the crack.

As regards the filter dimensions a common practice is that presented in Figure 4-56 where the diaphragm not only surrounds the conduit but extends vertically not less than 3 times the diameter of the conduit or to a level equal to or greater than the normal water level in order to capture all the cracks that may form as a result of the presence of the conduit.

Figure 4-56 show a graphic representation of the minimum recommended dimensions for this type of works, while Figure 4-54 shows the construction process of a diaphragm in an earthen dam. The dimensions and layout shown must be planned as per the dam section of each project. The diaphragm not only has to arrest migration of soil particles due to possible leaks around the conduit but, also due to any leaks as a consequence of the construction of the conduit, from hydraulic fractures which may occur in the dam body in that section.



Figure 4-55: Filter diaphragm trench (NRCS Ch.45, 2007)

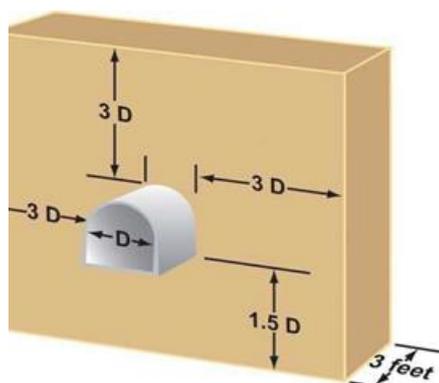


Figure 4-56: Typical configuration for a filter diaphragm. (FEMA, 2005)

The dimensions transverse to the conduit must cover, at a minimum, the special compaction zone established during the construction of the dam.

If the only postulated flow path is along the contact between earth fill and the conduit, the filter diaphragm may not need to extend far from the conduit. As an example, some agencies only use an 18-inch-thick filter diaphragm, which is similar to a filter collar. In other cases, the embankment dam may be subject to hydraulic fracture in areas that are well above and on both sides of the conveyance structures. In the absence of a chimney filter, the filter diaphragm may then need to be much wider and taller than the conveyance structures dimensions to intercept those cracks.

## 4.5 Lessons

In this section some examples of works that, to some extent, have affected the hydraulic safety of the outlet work in particular and of the dam in general, will be presented.

The following is a summary of three (3) representative cases of failures or incidents. They are: Turimiquire Dam (Venezuela); Anita Dam (U.S.A.) and Lawn Lake Dam (U.S.A.)

### 4.5.1 Turimiquire Dam (1988)

**Location:** Estado Sucre, Venezuela on Turimiquire River.

**Purpose:** Water Supply, Irrigation

**Dam:** Concrete Face Rockfill Dam, 111 m high

**Dates:** Built during the period 1976-1988, incident in 1988

**Outlet Works:**

**Intake:** Vertical Intake Tower 109 m high; . 6 inlets for water supply, sediment discharge and ecological purposes.

**Control structure:** In Tower Intake: Butterfly valves of 2.17 m diameter. At terminal structure: Fixed cone valve 3.00 m in diameter (VHB3) for emptying the reservoir and fixed cone valve of 0.60 m diameter for ecological discharges.



Figure 4-57: 109 m high Tower Intake Turimiquire Dam

**Conveyance structure:** Concrete tunnel of 6.50 meters in diameter and 453.88 meters in length and steel pipe of 3.00 m meters in diameter and 183.62 meters in length (See Figure 4-57 and Figure 4-58).

**Incident:** The first filling process of the reservoir was carried out by controlling the reservoir levels with partial opening of the 3.00m Howell-Bunger valve located at the lower end of this outlet work. In November 1988 under a hydraulic load of 78 meters, the drain valve was opened full and, as a result, the following was observed:

- Visible swinging of tower intake
- Vibration of tower control devices

- Sudden and abnormal closing of the feedback level No. 4. Subsequently, it was not possible to operate this valve.
- There was a sudden current of air flow inside the tower while the reservoir was emptying
- Air flow currents inside the tower generated damage to stairs and other accessories of the tower

Subsequent analysis of the situation established that the installed flow control devices did not meet the specifications of the original design and, at the time of the total opening of the bottom outlet valve, the hydraulic flow control temporarily moved from downstream (VHB3) to the tower intake since the discharge capacity of it was much higher than the discharge capacity of the butterfly valves in the tower intakes.

From this moment a cycle began where ~~in~~ due to the higher discharge capacity of the downstream valve the tower intakes got emptied and admitted a large amount of air into the tunnel. This was followed by increase in the discharge capacity of water admitted from the lower levels of the intake and the water levels internally increased in



Figure 4-58: Drawdown & Ecological Valve Turimiquire Dam

the tower intakes. Subsequently, the VHB3 with a greater hydraulic load, increased the discharge and emptied the tower again completing the cycle.

The oscillation problems in the tower resulted from the simultaneous entry of flow

jets of varying magnitude and radial arrangement.

The failure of the butterfly valve was due to the imbalance of loads between the level of the reservoir and the discharge level of the emptying tunnel (313-235 meters) and the vibrations in the devices due to the excess velocity of air that occurred in the valves.

Once the event passed, and analyzing the status of the outlet tower after the incident, it was decided that the opening of the VHB3 should be restricted to a maximum of 60% and to develop a new operation rule.

**Lessons:** The hydraulic study of the intake works and, especially those where the flow rates and pressures are significant, must be carried out in detail and the specifications of all the devices that are installed should be worked out precisely. Changes during construction must also be analyzed and, taking into account all operating conditions, its operation should be modeled in such a way that it is viable for all expected conditions and, in particular, for the extreme conditions that develop due to the drawdown operations.

REFER TO APPENDIX A

Failure modes associated with valve and gate malfunction are presented (FM-22 & FM-23)

#### 4.5.2 Anita Dam (1997)

**Location:** Montana, 22 miles north of Chinook, Montana, U.S.A. on unnamed tributary of the East Fork of Battle Creek.

**Purpose:** Flood control

**Dam:** Embankment Dam, Homogeneous fill with a layer of upstream riprap. 36 feet (11m) high.

**Dates:** Built in 1996, incident in 1997

**Outlet Works:** Drop Inlet 36” steel outlet conduit with concrete anti-seep collars.

**Incident:** Between March 22 and 26 of the year 1997 an unusual volume of water from melted snow filled the reservoir. On the morning of March 26, 1997 an important leak was observed on one side of the discharge conduit that alerted operators and, in general, all the residents near the dam were warned of a possible evacuation.

The immediate presence of the emergency teams, the National Guard and a Type II incident Command team made it possible to continuously monitor the development of the incident.



Figure 4-59: Outflow during failure. 26Th March, 1997 (ASDSO)



Figure 4-60: Terminal structure seen from downstream. (ASDSO)



Figure 4-61: Antiseep collars in outlet conduit during construction. (ASDSO)

The leaks continued to increase reaching an estimated 11,000 liters/sec (400 cusec), a value greater than the maximum conduit discharge capacity. Simultaneously, vortex formation was observed in the reservoir.

36 hours after the incident was perceived, the reservoir of almost 980,000 cubic meters was drained. See Figure 4-59 to 4-63.

Once the reservoir was emptied and the information related to the construction of the reservoir was checked, it could be verified that:

- During the incident, a gap was formed from the foot of the slope upstream of the reservoir to the foot of the slope downstream following the outer face of the conduit.
- The soil material used in the body of the dam were low plasticity clays with dispersive characteristics.
- According to construction reports, soil compaction in the immediate vicinity of the conveyance structures and between collars was performed with manual compaction equipment.
- The material used to fill the base of the conveyance structures and served as a support for it was a mixture of high workability cement soil.



Figure 4-62: Manual compaction around conduit (ASDSO)

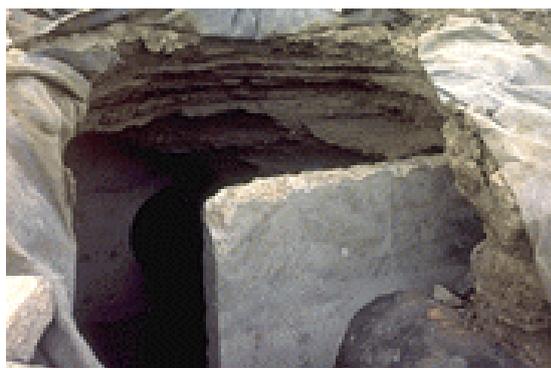


Figure 4-63: View of Upstream end of conduit after failure. (ASDSO)

- The low temperatures that occurred during the winter season formed lenses of frozen material that, when melted, provided a plane of failure that allowed the initial seepage/leakage and the process of hydraulic fracture that was observed later
- There was no presence of filter material in the area near the discharge of the conveyance structures.

It is considered that in Anita Dam a combination of reasons wherein the most important were the hydraulic fracture generated by the failure due to poor compaction of the soils around the conveyance structures combined with a dispersive material not suitable for use as filling material for the dam and insufficient leakage protection measures.

REFER APPENDIX A

Where failure modes by piping in embankment dams in contact with the conveyance structures of outlet are presented

**Lesson:** The collars do not constitute a solution to this problem and, on the contrary, they make the compaction around them very difficult.

The use of clays with dispersive characteristics should be handled with great caution and, in case of not having other alternative material, it is necessary to do treatments with lime or any other suitable

material that stabilizes the soils and prevents their reaction to changes in pore pressure and, in general, of the conditions of humidity in the ground.

Design of new conduits/outlet works in earth embankments should not make use of collars, their external walls should have mild slope (not vertical) and should be covered with materials that allow a better degree of compaction and integration between the materials.

In the slope downstream of the dams it is convenient to construct a filter diaphragm that captures the migrating soil particles and properly disposes of any seepage/leakage around the buried conduit.

#### 4.5.3 Lawn Lake Dam (1982)

**Location:** Rocky Mountain National Park approximately 10 miles upstream from Estes Park, Colorado, U.S.A. on Roaring Fork River.

**Purpose:** Irrigation

**Dam:** Embankment Dam, Homogeneous fill with a layer of upstream riprap. 26 feet (8m) high.

**Dates:** Built in 1903, incident in 1982

**Outlet Works:** 36-inch diameter steel pipe with a gate control valve situated at center of the embankment dam.

**Incident:** On the morning of July 15, 1982, the dam's embankment a breach formed having a trapezoidal section 8.50 meter high, with a width of 16.8 meters at base and 29.6 meters at the top. During the failure, 830,000 m<sup>3</sup> of water were released with an estimated maximum discharge of 510 m<sup>3</sup>/s (USGS, 1982). See Figure 4-64 to Figure 4-66.

A subsequent evaluation of the material that constituted the embankment revealed that the predominant material was poorly graded silty sand with a high percentage of organic materials.

Although the failure process could not be observed, subsequent investigations point to a process that began with a large leak of pressurized water that occurred at the joint between the steel conduit and the gate valve used to control the flow rate in this outlet work. At the time of failure, the pressure in the conduit was maximum since the reservoir was full and the valve was closed.

The water leakage began a process of progressive erosion that ended up generating piping up to the storage reservoir and, once it reached there, the hydraulic load of the reservoir, and the erosive power of the reservoir water generated the final breach section.

The abrupt discharge of flow created a flood wave that was dragging everything in its path causing the fall of trees and infrastructure. (Figure 4-64)

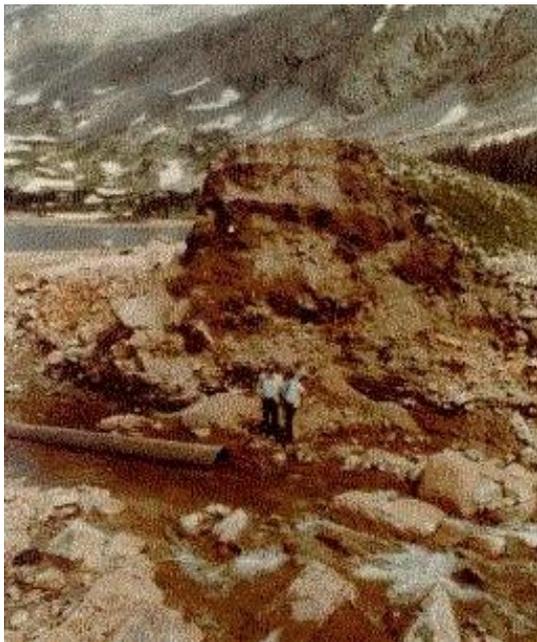


Figure 4-64: Lawn Lake Breach and Conduit (ASDSO)

The Cascade Lake Dam (4 m high concrete gravity), located almost 10 kilometers downstream of Lawn Lake, was overtopped and it finally collapsed as a result of the rising thrust. (Figure 4-65) Thus, a new flood wave occurred that continued until it

reached Estes Park, about 16 kilometers downstream of Lawn Lake.

The final consequences of the rupture of this dam were measured at more than \$ 35 million for losses to businesses and damage to property and the unfortunate death of 3 people. It is worth noting that the timely warning of the movement of the floodwave saved the lives of many campers who enjoyed nature in the summer of 1982.

As a result of the failure, the Roaring River and Fall River were widened and erosive processes occurred in the channel that undermined its bottom between 1.50 and 15 meters. The deposit of materials in the valley of the River Fall is estimated at 2,300,000 m<sup>3</sup> and in its route the flow reached such power that it moves rocks of 450 ton of weight.

**Lesson:** The failure was caused by a combination of internal erosion and backward erosion/piping generated by a pressure leak from the conduit.



Figure 4-65: Cascade Dam overtopping. July 15th 1982 (ASDSO)

The construction of pressurized ducts under embankment dams should be avoided and, if their execution is absolutely necessary, all necessary precautions should be taken to increase safety levels with emphasis on the design or protection of the joints and, in case if possible, incorporate a larger pipeline that allows permanent monitoring.

## Chapter 5. ENERGY DISSIPATORS

### 5.1 Overview

A safe hydraulic operation of the appurtenant works (spillway and outlet works) of a dam must be ensured from the inlet at the reservoir to its exit in the downstream channel. In describing hydraulic safety (Chapter 1), it was highlighted by Soriano and Escuder, 2008 that:

- a) The operation of spillways and outlet works must be satisfactory for the full range of discharges, including ordinary and extraordinary flows.
- b) The conveyance and re-integration of the flow to the downstream river or watercourse, without causing any damages, are key functions of appurtenant works.

In previous Chapters 3 and 4, spillway and outlet works were covered up to the end of the conveyance structures; this chapter deals with terminal and exit works whose function is to return the flow suitably back to the river.

Two main aspects which are of particular importance in hydraulic performance of these terminal works are:

- a) Dealing with flow with high velocity and turbulence and high energy content.
- b) Conversely, restoring the flow to the river without causing damages to the dam, and to close-by hydraulic works, river/ watercourse and to the nearby environment.

These two aspects have been a matter of concern for researchers and designers, since they constitute a failure mode (FM) of the spillway globally named “failure to accommodate flow with high energy content”. This FM can be subdivided according to

several hazards or hydraulic loads acting on the spillway downstream of the control structure; it gathers velocity that may induce malfunctioning, damages to fragile elements and structural failure of components. These hazards have been the cause of major incidents with temporal loss of the spillway and significant impacts to reservoir-dam safety. Figure 5-1 shows a typical high velocity flow (high energy content) at the terminal structure.

From the point of view of rehabilitation, it is important to note that the increase in flood flow (SPF, IDF or PMF) results in an increase in maximum water level and specific discharge (as presented in Chapters 2 and 3); this may alter the hydraulic operation of each component, especially the structures at the downstream end: energy dissipator, plunge pool in the receiving water body and exit channel. The modified flow behavior induces an increase in potential hydraulic hazard due to larger dynamic loads and subsequent hydraulic and structural responses.



Figure 5-1: Typical flow with high energy content

In order to describe a flow with high energy content, some authors and researchers use as an index of hydraulic severity, the power of the flow (gross power in MW or unit power in MW/m, by meter of width of terminal structure at its inlet point). In a dam of intermediate height with an acting water head of 25 meters and spillway with a discharge capacity of 1,000 m<sup>3</sup>/s, the flow reaches the terminal structure with a gross power (energy flux per unit time) of 250 MW approximately; as head and flow increases, flow power can be enormous. (Table 5-1 and Figure 5-2).

Hydraulic characteristic	Figure
Design discharge	8,467 m <sup>3</sup> /s
Controlled by 5 radial gates of width	14 m
Unit discharge at the inlet of chute	100 m <sup>3</sup> /s/m
Unit discharge at the inlet of terminal structure	70 m <sup>3</sup> /s/m
Hydraulic head on basin's floor	113.2 m
Gross power of flow = (9.81 * 8467*113.2)/1000 = 9400 MW	9,400 MW
Unit power at the inlet of terminal structure	78 MW/m
Velocity of flow at the inlet of terminal structure	42 m/s
Kinetic energy head at the inlet of terminal structure	90 m

Table 5-1:Hydraulic characteristics of spillway of Ramganga Dam

USACE (1990) has reported operation experiences of energy dissipators for unit power in the range 6 to 170 MW/m, including several cases with different types and severity of structural damages for values of the flow unit power greater than 25 MW/m, but especially for values greater than 60 MW/m. Semenkov (1979), in a study of 400 different type of spillways, chooses the gross power as a characteristic index covering a range from 1 to 100,000 MW. Even though the above power is not used internationally to qualify spillways, especially energy dissipators, figures above 10,000 MW and 50 MW/m could be the threshold to define severity of flow. Experiences from operative spillways and advances in research on energy dissipators performance through mathematical (Computational Fluid Dynamics, CFD) and physical models have helped to design energy dissipators with higher values of unit power of flow and unit discharge. Chanson (2015) emphasizes the importance of understanding the figure of power of the flow and the complexity of the hydraulic behavior of energy dissipators due to “the physical processes taking place and to the structural challenges”.

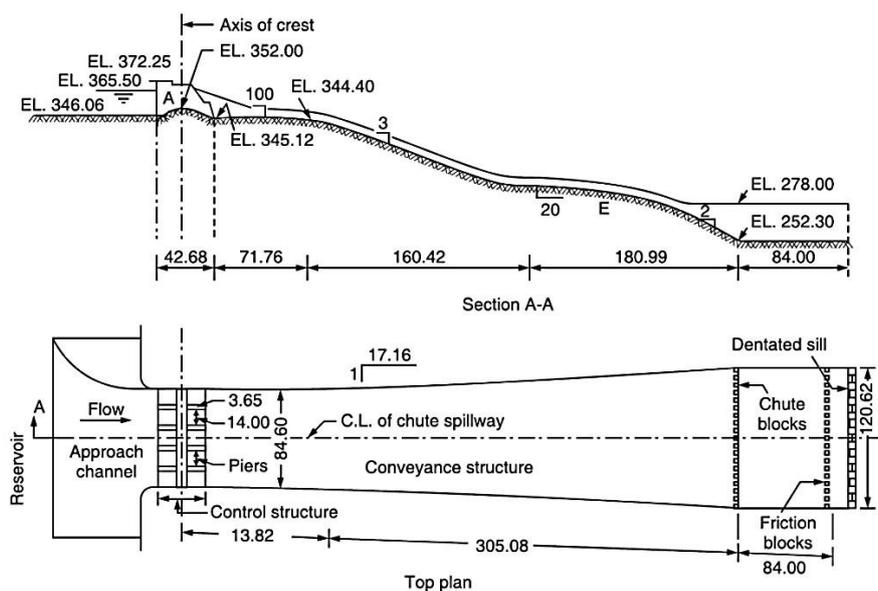


Figure 5-2: Spillway of Ramganga Dam (Uttarakhand, India) (Dankekar and Sharma, 2013)

### Number of Spillways VS Gross Power

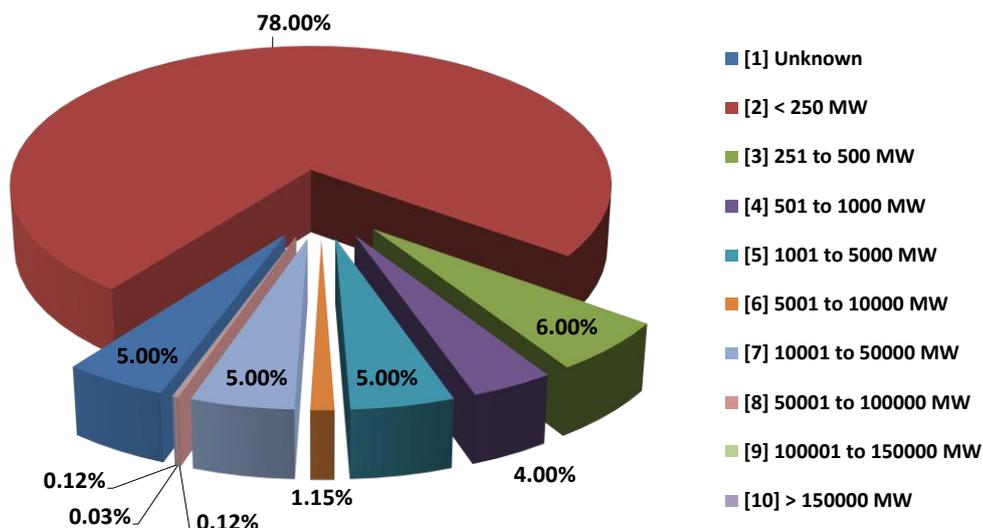


Figure 5-3: Spillways in India’s dams (NRLD) – Distribution of spillways by Gross Power

A review of spillway’s data from National Register of Dams (NRLD, CWC, 2019) allowed in deriving a frequency distribution of Gross Power as presented in Figures 5-3 and 5-4, the first for the total number of dams in India and the second for the sample (70 dams) named as “Dams of National Importance”. Both figures give an idea of distribution of operative condition of terminal structures at India’s dams, and also highlight that about 1% (68 dams) are above

the threshold of 10,000 MW, which includes 40% (28) of the dams of national importance group.

As mentioned in previous chapters, structural and hydraulic aspects encompass total safety of an appurtenance work within the dam-reservoir system. Safety of terminal structures (energy dissipators, plunge pools and exit channel) not only has to do with hydraulic behavior of the works, and its

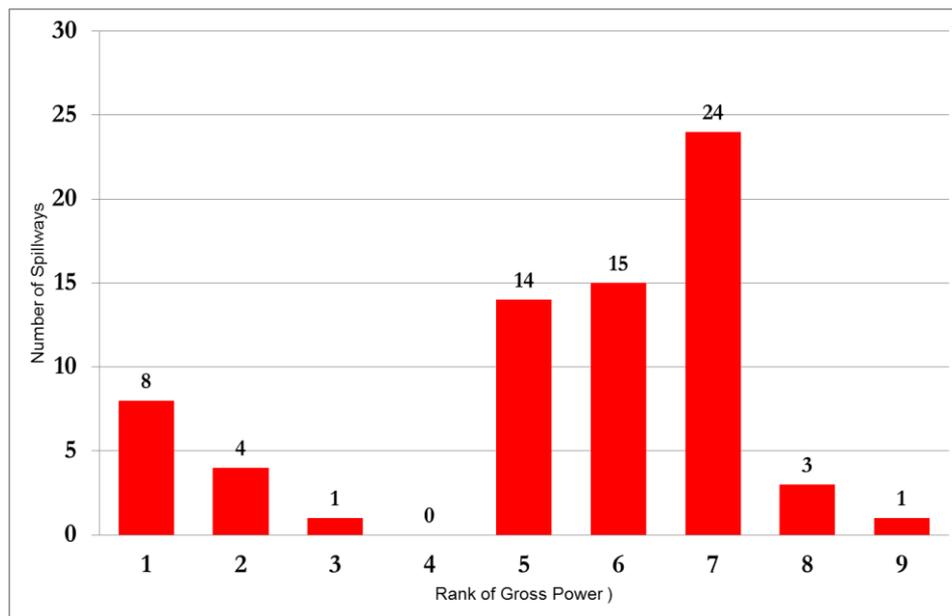


Figure 5-4: Spillways in India’s dams (NRLD) - “70 Dams of National Importance”(For rank of Gross Power refer Figure 5-3.

structural stability but also with characteristics and response of the receiving river environment (topography, geology, ecology, etc).

As this manual is part of CWC series of Guidelines and Manuals, some topics related with these component of appurtenant works are also covered in other documents such as the “Manual for Assessing the Structural Safety of Dams and the “Manual for Rehabilitation of Existing Dams”. Other related CWC documents are: “Guidelines for Safety Inspection of Dams”, “Guidelines for Assessing and managing Risks associated with Dams” and “Guidelines for Classifying the Hazard Potential of Dams”.

This chapter, does not cover aspects related to the hydraulic design and construction of the terminal structures, the user is referred to the extensive technical literature on this subject, especially that in ICOLD, USBR, USACE, FEMA, several authors and research’s institutions from India and many others countries and in Bureau of Indian Standards (BIS). The scope of this chapter is limited to existing dams; it embraces evaluation of existing energy dissipators structures and their operation, potential hydraulic hazards on various elements or equipment, modes of malfunctioning or failure (FM). Rehabilitation of these components of the spillway encompass measures for repairing or upgrading non-functional structures and managing their risk to accepted or tolerable level.

Appendices A, B, D and E of Volume 2 of the present Manual shows related aspects of interest as: Failures Modes Identification, Cases of Study, Hydraulic Modeling and Hydromechanical equipment.

## 5.2 Definition and Function

The terminal and exit structures are fundamental components of any spillway. Since the mass of water moves from a high level in the reservoir to a lower level i.e. the riverbed level downstream, there is always the

need to cope with flow having higher energy content than natural river conditions. It is essential that the energy dissipater should perform well for the entire range of expected discharges and that it meets the requirement of “reintegrating the flow to the downstream watercourse in appropriate conditions”. If this requirement is not fulfilled several adverse responses can be expected, the worst being the generation of an erosion process of the riverbed with severe damage to the spillway itself, to other near structures, to the dam and to the river environment (bed and banks).

The following terms will be frequently referred to in this chapter (see BIS and Appendix F Glossary of this Manual):

- Energy dissipator: Any device constructed in a waterway to reduce or destroy the energy of fast-flowing water (also referred to as Terminal Structure).
- Energy dissipating valve: A generic term used to describe those regulating valves that are designed to dissipate flow energy. For terminal structures this valve is located at the downstream end. In many cases, it discharges freely into the atmosphere.
- Exit, outlet or discharge channel (downstream): Conveyance of water from the terminal structure to the river or stream.
- Plunge basin: An artificially created deep pool that dissipates the kinetic energy of free-falling water before being returned to the downstream channel (also called Plunge Pool).
- Stilling basin: A basin constructed so as to dissipate energy of fast-flowing water by means of a hydraulic jump.
- Solid roller or slotted roller bucket: Hydraulic structure provided to dissipate energy of fast-flowing water by interaction of the water rollers formed. A high tail water is required for its functioning.

- Flip or trajectory bucket: Hydraulic structure that changes the direction of the flow and directs it at a certain distance downstream where the energy is dissipated by inter-action with the receiving water body and the ground. These structures operates with tailwater either lower or slightly higher than the bucket lip elevation.
- Tailwater: The level of water in the receiving channel (exit channel) immediately downstream of the terminal structure of the spillway. The tail water elevation varies with discharge.

The energy dissipation in appurtenant works can be achieved by the following ways:

- Turbulence created by change of the hydraulic regime: sudden change from rapid flow (high velocity) to tranquil flow (low velocity) with an abrupt rise of water level, wave and surge formation at surface with high air entrainment and strong eddy formation. This hydraulic phenomenon is called hydraulic jump.
- Water diffusion in a pool: a high velocity flow or jet entering into a mass of water (cushion).
- Inter-action of water rollers: it is referred to inter-action of the two rollers formed in a solid roller bucket/slotted roller bucket - one in the bucket and the other outside it on the downstream region .
- Water impact with the ground or tail water: a jet or trajectory from a flip or trajectory bucket falling sufficiently away from the bucket and, by its impact with the ground/tail water resulting in scour.
- Water dispersion in the air: discharge of water from some valves as a diverging jet into the atmosphere with high water-air mixing.

- Water impact or friction in a specially designed structure or friction/form resistance to flow along a chute.

Figures 5-5 to 5-9 show examples of energy dissipation.

The hydraulic variables that participate in the process of energy dissipation in cases of surface flow in spillways and outlet works, are:

- Flow characteristics: Velocity ( $V$ , m/s) and Froude number ( $F$ )
- Intensity of discharge or specific discharge ( $q$ ,  $m^3/s/m$ )
- Tail water level ( $TW$ , m)

For pressure flow as in some cases of outlet works, the hydraulic variables are: energy head upstream of discharge valve, discharge and flow velocity.



Figure 5-5: Energy dissipation by turbulence due to Hydraulic Jump (Stilling basin FEMA, 2010).

The local geology must guarantee stability of exit channel or plunge pool due to action of flow velocity and impact of the water jet. If necessary, protection works d/s of the energy dissipation arrangement may be considered to avoid erosion. The tailwater level ( $TW$ ) plays a key role in ensuring the proper

functioning of the terminal structure. The tail water level varies with discharge and depends on the topographical and hydraulic characteristics of the downstream waterway/river (channel's section, slope, roughness). The Tail Water Rating Curve should be developed with "best possible" data or by direct river measurements with great care.



Figure 5-6: Energy dissipation by dispersion in the air and impact (flip bucket).



Figure 5-7: Energy dissipation by impact flip bucket (EDALCA, Venezuela)



Figure 5-8: Energy dissipation by impact on a stepped surface



Figure 5-9: Energy dissipation by dispersion of water in air (emerging flow from a valve) (FEMA, 2010)

### 5.3 Classification of Energy Dissipators for Spillways and Outlet Works

The energy dissipators for spillways and outlet works can be classified according to hydraulic action, mode of energy dissipation, and geometry/type of terminal structure (Khatsuria, 2005); these being the most used factors in technical references. In this chapter only energy dissipators for large dams are presented, particularly those used in spillways for embankment dams and concrete gravity dams as they are the most common types of dams in India. Other types of terminal structure are only mentioned and not covered in detail; for details there is an ample technical literature available which can be referred to.

The types of energy dissipators used for ungated and gated spillways are similar; however, for tunnel spillways, even though the principles are the principles of energy dissipation are the same, there are some differences due to their geometry and hydraulic functioning. For outlet works there may be specific structures for this purpose; special type of valves are sometimes used in cases where the steel conduits are provided as outlets. In many projects, energy dissipation for both spillway and outlet works is combined in one structure, for example, intermediate or bottom sluices or valve discharging in the terminal structure of a surface spillway.

The energy dissipators for large dams can be grouped as follows:

For surface flow spillways:

- Stilling basins
- Bucket types:
  - Roller bucket – Solid Roller/Slotted Roller.
  - Flip bucket or free jet bucket (flow deflector).
- Other types for particular cases:
  - Impact type for free over fall spillway (free waterfall).
  - Non-conventional or variations of previous types tested in physical models at hydraulic laboratories for tunnel and orifice spillways (surface or pressure flow):
  - Steeped chute: is a special structure which combine conveyance and dissipation functions, for concrete gravity dams (specially RCC) and for embankment dam overtopping (as presented in Chapter 3). This can be used as an only dissipator or in combination with conventional energy dissipator at its toe, to cope with residual energy in the flow.

For outlet works:

- Stilling basins.
- Bucket type:
  - Flip bucket or free jet bucket (flow deflector).
- Impact type.
- Special valves.

The most common types of terminal structures in embankment dams for a central spillway or on an abutment or at any other location, are stilling basins and flip bucket (flow deflector). For concrete gravity dams with spillways in the river bed, both stilling basin and bucket types are common. These

days slotted roller bucket is avoided as its teeth are prone to continuous damages. In RCC gravity dams, a typical solution is stepped chute with or without stilling basin at dam's toe. In arch dams, the dam geometry may be used to incorporate an intermediate flip bucket if the spillway is located in the dam itself or impact type as in spillway water fall. ; in some cases, the spillway may be located separately from the arch dam e.g. the spillway for the Idukki arch dam is not located in this dam but is located in Cheruthoni Concrete Gravity dam which is part of the Idukki complex having a common reservoir. In some technical references a head of 100 to 120 m is considered an upper limit to use stilling basin; though, there are examples of use of stilling basins for much greater heads like Tehri dam (energy head greater than 200 m), Bhakra dam and others.

For outlet works in a dam, the most common types of energy dissipators are stilling basins, free jet bucket type and special discharge valves.

In India most dams spillways include stilling basins and bucket type energy dissipators; for sluices located in spillway, normally no separate energy dissipation arrangement is provided and the energy dissipater for the spillway is considered adequate for the sluice also.

For bottom type spillway: orifice, breast wall and bottom outlet, energy dissipation is challenge due to: high flow concentration, heavy load of sediments passing through the structure and in many cases, discharge is very closed to the river bed level. In this cases, long flat slabs and submerged buckets have been used. There are several examples of this type of energy dissipators in dams located at Himalayan region.

The common types of terminal structures and exit works especially those used in Indian Dams are described below. The user of the manual may refer to various other tech-

nical references available on the subject also including Bureau of Indian Standard codes/guidelines. In particular, it is necessary to verify the design of energy dissipators in physical models in view of the complexity of the flow.

REFER TO APPENDIX D

For Hydraulic Model Studies for Dam Safety Assessment

### 5.3.1 Stilling basin

This type of energy dissipator consists of a basin, usually with a rectangular section and a robust reinforced concrete structures (bottom slab and walls), located downstream of a spillway or an outlet work, in which the kinetic energy of rapidly flowing water is dissipated by the formation of a hydraulic jump.

#### 10.3.1.1 Basic hydraulic functioning

In a stilling basin, energy dissipation occurs by the development of macro turbulence and internal friction, as a result of the abrupt change of channel slope flow regime suffers a sudden change from high velocity (supercritical) to low velocity (subcritical). This hydraulic feature is called hydraulic jump; its use is wide-spread and is a reliable way to dissipate energy. Usually the bottom of the basin is horizontal but, in some cases, could be sloped, also cross section is usually rectangular but there are trapezoidal basins also. Figure 5-10 shows a schematic profile of a hydraulic jump and energy balance.

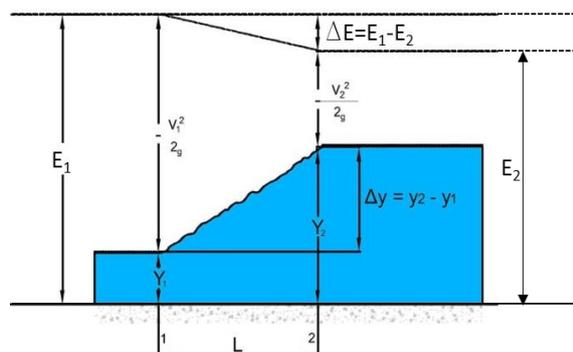


Figure 5-10: Hydraulic jump profile

The hydraulic parameters of a hydraulic jump for flow with an intensity or specific of discharge “q” are:

- At entrance section (end of chute/ spillway d/s glacis), supercritical flow:
  - Energy ( $E_1$ ) = [Maximum Water Level at Reservoir] – [Energy losses along the chute] (m) (measured from the bottom of basin inlet)
  - Flow velocity ( $V_1$ ) (m/s)
  - Water depth ( $y_1$ ) (m)
  - Froude number ( $Fr_1$ ) meter formula  

$$Fr_1 = \frac{V_1}{\sqrt{gy_1}}$$
- At exit section (end of the jump), subcritical flow:
  - Energy ( $E_2$ ) =  $E_1 - \Delta E$  (m)
  - Flow velocity ( $V_2$ ) (m/s)
  - Water depth ( $y_2$ ) (m)

The jump has a length “L” (m) from section with clear change in water surface to a downstream section where turbulence has been damped and water surface is almost smooth.  $\Delta E$  is the energy loss between sections 1 and 2, which varies with the intensity or type of jump and can be up to 75% of  $E_1$ .  $\Delta y$  is the increase of water depth which depends on the type of jump. The Froude number  $Fr_1$  is the parameter used to define type of jump from Pre-jump (with low energy loss) to good-steady jump (with high energy loss). For large dam spillways, incoming flow in stilling basins has Froude number greater than 4.5 to more than 10.

As per the hydraulic jump equation (Bé langer), for a rectangular basin with horizontal floor, the depth downstream ( $y_2$ ) in terms of the depth at the inlet section ( $y_1$ ), is given as:

$$\frac{y_2}{y_1} = \frac{1}{2} \left[ \sqrt{1 + 8Fr_1^2} - 1 \right]$$

The flow depth at section 2 is named “conjugate depth” and is used to evaluate the location and stability of the jump within the basin.

This depth ( $y_2$ ) is not the tailwater depth. The relationship between tail water depth (or level) and discharge, the Tail water rating curve, is a hydraulic characteristic of the downstream channel or river, so it is the natural water level for each discharge. The hydraulic jump formation requires that the tail water depth to be equal or some greater to  $y_2$  for that particular discharge.

In laboratory conditions a fully developed and stable hydraulic jump, called Free (or clear) Jump, can be generated such as  $y_2$  equals TW; however, in site conditions, these water levels do not match, furthermore the relationship between them varies, so  $TW > y_2$  for a flow discharge and  $TW < y_2$  for others, including larger flows. The effect of TW is interpreted as a force that acts and holds the jump in a location, then if  $TW < y_2$  the jump moves downstream (sweep out of hydraulic jump) and if  $TW > y_2$  the jump moves upwards to the chute (submerged or drowned jump). In a stable condition, TW holds the jump in a position (inside the basin) so  $E_2$  equals energy at exit channel.

REFER TO APPENDIX A

Unstability of the Hydraulic Jump FM 14

In the case of a stilling basin, the required TW is achieved with a combination of downstream water depth (channel or river) and invert (apron) elevation of the basin's floor, but since TW can vary due to changes in exit channel (aggradation or degradation) a safety factor has to be used in water level. In some cases, a weir is located downstream to fix and control water level for the jump. In an existing stilling basin, especially if the design flood has increased, the formation of the jump and the adequacy of the length of the stilling basin for higher discharges must

be checked to evaluate the current performance of the basin. This analysis encompasses the following aspects:

- Work out the tailwater rating curve (stage-discharge relationship) for exit or natural channel. This curve requires a water surface profile analysis from a control section downstream at the river up to the outlet section of stilling basin.
- Work out conjugate depth curve (for basin width), using the jump equation for several discharges.
- Check of stability of the hydraulic jump.
- Verification of condition of the hydraulic jump: stable, drowned or sweep out, to define hazardous actions on structure and exit channel.

Figure 5-11 shows the relation between conjugate depth and TW for a particular discharge, in an existing stilling basin and Figure 5-12 presents an example of basin with a downstream weir.

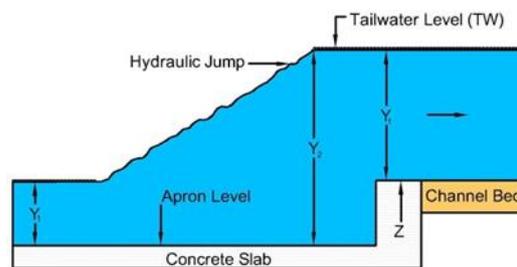


Figure 5-11: Relation between conjugate depth and tailwater depth



Figure 5-12: Typical stilling basin (with downstream weir to control TW)

The key evaluation of hydraulic performance, for a particular discharge, would be:

- If TW above river bed =  $y_t$  (from tail water rating curve) so  $y_2 > y_t$  but available TW is measured from the floor of stilling basin including Z (depth of apron below channel bed) then the current TW =  $y_t + Z$ , thus conjugate depth water level (Apron level +  $y_2$ ) agrees with TW level.
- If  $y_2 > TW + Z$ , then the hydraulic jump sweep out occurs. Thus the level of the apron in that case is too high for adequate performance for that discharge. On the contrary, if  $TW + Z > y_2$ , hydraulic jump moves upwards on the chute, so apron setting is too deep. Under this situation a drowned hydraulic jump occurs.

The length of a free hydraulic jump can be estimated by several equations. Two common equations, for usual range of Froude number in spillways, are as under:

$$L = 6.9 \cdot (Y_2 - Y_1) \text{ or } L = 6 \cdot Y_2$$

This free hydraulic jump length is significant and efficiency is optimum but the basin length is generally too long. Research on physical models has allowed defining of forced jump in basins with ancillary elements (chute blocks, basin blocks, end sill) at the bottom that contribute to dissipation by the effect of local turbulence and impact of the flow, with resulting less length of the jump. These tested basin lengths are used in most of the large dams.

REFER TO APPENDIX D

Hydraulic Modeling

Due to the complexity of the hydraulic behavior of high velocity flow and hydraulic jump, physical modeling as a tool for verification is invariably carried out, not only during the design stage for a new structure

but also to evaluate existing structures for discharges other than the discharge for which it has been designed. Figure 5-13 shows a typical model of a stilling basin.



Figure 5-13: Typical model of stilling basin with drowned jump (FEMA, 2010)

### 10.3.1.2 Types of stilling basins

#### a) Stilling basins for spillways

A stilling basin can have a horizontal flat apron or a sloping apron for developing a complete free hydraulic jump or an apron with ancillary elements as chute blocks, baffle blocks, and an end sill at the exit of the basin, to force the hydraulic jump. There are standards designs based on flow parameters: Froude number, discharge intensity ( $m^3/s/m$ ), and velocity of incoming flow ( $m/s$ ). A series of standard stilling basins have been designed or presented by USBR and USACE in USA, Bureau of Indian Standards and other organizations. For special cases, stilling basin may be designed or verified by using physical models.

As mentioned, the ancillary elements are used to produce turbulence, improving energy dissipation, to shorten the basin length. Their characteristics, shape, width, height and location on the apron are key to ensuring a good jump control and energy dissipation efficiency. The design of these elements is based on tests carried out on hydraulic models for different flow's energy content and entrance velocity. The use of these elements, especially, the baffle blocks, is limited by velocity of flow due to the high potential of damage that they may suffer in the process of energy dissipation. The inclusion of a sill at the end of the basin is required to direct the flow and to reduce

scouring d/s of the basin. Figures 5-14 and 5-15 show typical stilling basins for large dams: the first one with horizontal apron and full jump length (usually an end sill is included) and the second, with ancillary elements (chute blocks and dentated end sill, with tested dimensions).

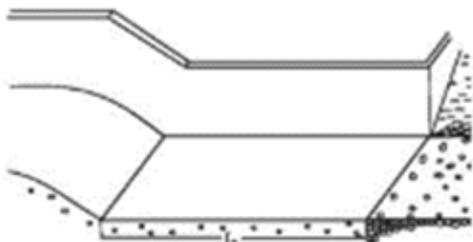


Figure 5-14: Stilling basin with horizontal apron without ancillary elements

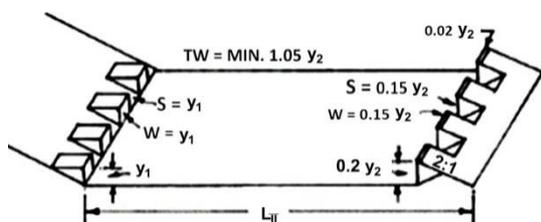


Figure 5-15: Stilling basin USBR Type II (Chow, VT, 1981)

An important aspect related with basin performance and its hydraulic safety is the required height of training walls to avoid splash and overtopping. In order to fix this height, a freeboard must be added to the conjugate depth ( $y_2$ ). Figure 5-16 indicates the required freeboard above the water level after the jump. In cases of existing stilling basin whose capacity be increased, the available freeboard must be checked.

USBR presents the following expression to fix the freeboard, it was established from physical models of their standard basins. For special cases, required freeboard of training wall has to be defined in hydraulic models.

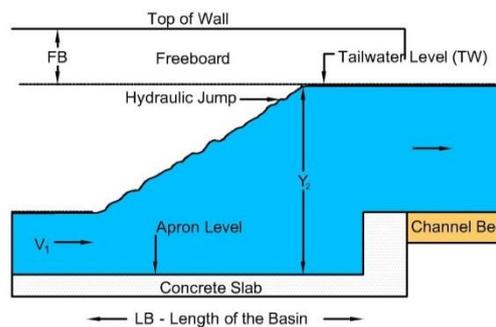


Figure 5-16: Stilling basin's freeboard

Freeboard for stilling basin's walls:

$$FB = 0.1(v_1 + y_2) \text{ (m) (USBR, in meters or feet)}$$

where,  $v_1$  = velocity of flow entering the basin (upstream of the jump) (m/s) and  $y_2$  = conjugate depth (m)

Other important aspect to be checked, especially in existing spillways, is the uniformity of the flow that reaches the basin and its distribution along the width of the chute. If the flow is not properly distributed, the efficiency of energy dissipation decreases. This case often occurs in gate-controlled spillways where the operation system may cause the flow to be non-evenly distributed in the channel. One solution, especially for cases of large structures and significant flow rates, is the partitioning of the conveyance and the basin by means of intermediate or splitters walls so that the discharges can be handled by independent channels and basins.

#### b) Stilling basins for outlet works

For outlet works, stilling basins are similar to those for spillways. However, side transitions and a parabolic curve for the bottom profile may be required to be provided from the end of the conduit/ tunnel to the beginning of the stilling basin, in order to conform to the width and the invert level of the stilling basin. The standard basins proposed for spillways are also used for outlet works according to flow's Froude number. Figure 5-17 shows a stilling basin for a tunnel.

For pressure flow or free jet emerging from valves, basins are also used, for some valves, being the basic arrangement a discharge directed downward at a certain angle (commonly around 25 to 30°) for better performance of the basin.



Figure 5-17: Stilling basin at the outlet of a tunnel (FEMA, 2010)

The aspects mentioned previously about evaluation of performance also applies to these basins for outlet works, as:

- Location of jump within the basin
- Tailwater required and its dependence of the level of apron.
- Freeboard requirements

- Distribution of flow across the width of the basin

For typical regulating and free discharge valve (Hollow jet, cylindrical water jet), common in many existing dams although less used at present, USBR developed a standard design of stilling basin, shows in Figure 5-18.

- c) Examples of stilling basins in spillways of India dams

In general, stilling basins are the most common type of energy dissipator for spillways used worldwide; in India also this is the case, generally as per standard designs (USBR and IS) and many tested and designed by physical models due to flow conditions. In the sample of 15 stilling basins included in IS-4997 (dams with more than 50 years in operation), unit discharge is in the range of 10 to 80 m<sup>3</sup>/s/m and Froude number from 4 to 11. Figures 5-19 to 5-23 show examples of stilling basins in spillways operating in Indian dams.

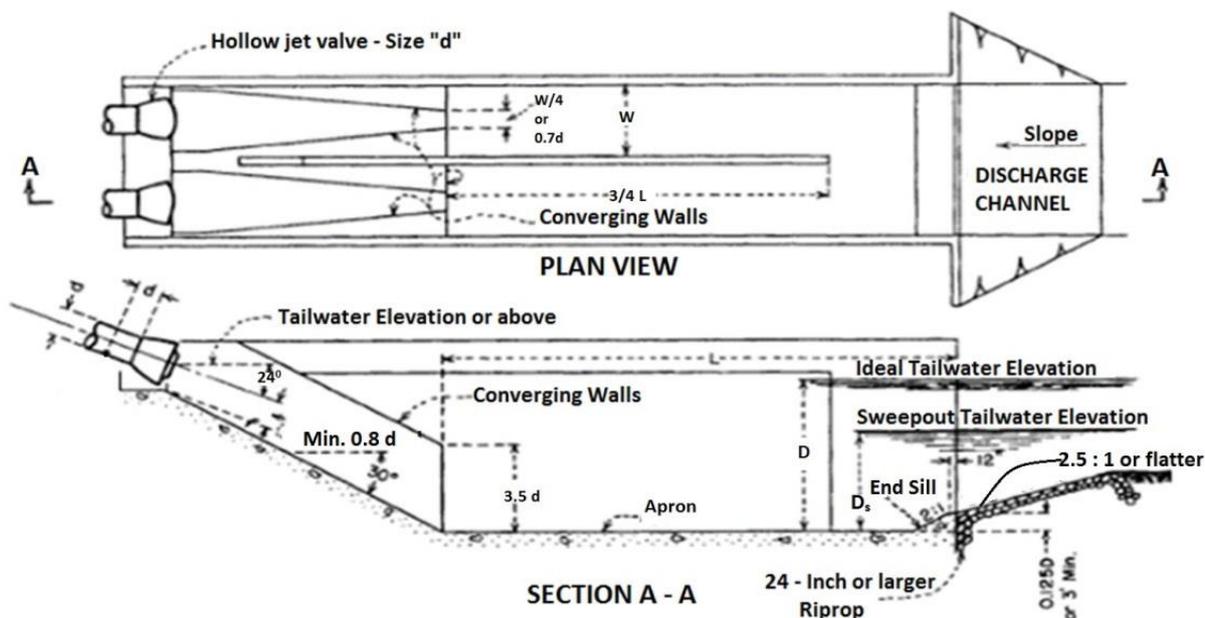


Figure 5-18: Stilling basin USBR VIII for Hollow-Jet valve (FEMA, 2010)



Figure 5-19: Koyna dam, height 103.2 m (Maharashtra)



Figure 5-20: Koteshwar dam, height 97.5 m (Uttarakhand)



Figure 5-21: Sardar Sarovar dam, height 163 m, discharge 84,950 m<sup>3</sup>/s (Gujarat)



Figure 5-22: Kalo dam, height 24 m (Odisha)



Figure 5-23: Tehri dam, height 260 m (Uttarakhand)

### 5.3.2 Roller bucket

USACE (1992) defines this terminal structure as: “A circular arc bucket tangent to the spillway face terminating with an upward

slope. This geometry when located at an appropriate depth below tailwater will produce hydraulic conditions consisting of a back roller having a horizontal axis above the bucket and a surge immediately downstream from the bucket”.

This type of energy dissipator is used when excessive tailwater depths exist for adequate energy dissipation by a hydraulic jump. This high tail water can be due to hydraulic characteristics of the river channel or foundation conditions that require siting the structure well below the channel. Also, this structure is cost effective for flow with high Froude number.

#### a) Basic hydraulic functioning

For an adequate energy dissipation by roller action, the setting of invert of this bucket has to be such that the required tail water

depths from the bucket invert to the tail water levels, for the entire range of discharges, is available. Also, it should perform well to the expected fluctuations of tail water. Tail water levels different from requirement may cause undesirable conditions such as: wave's formation, emerging jet surging downstream from the bucket without development of the two rollers. These circumstances, result in hydraulic instability, inadequate energy dissipation and erosion d/s of the bucket.

Further in a roller bucket which is performing well by way of formation of the two rollers and a good energy dissipation, still on account of the hydraulic action, river bed material d/s of the bucket can be drawn into the bucket by the d/s ground roller which is then partially swept out by the surface roller which is formed within the bucket. This causes damages in concrete due to abrasion by that bed material.

There are two types of roller buckets - solid and slotted, their hydraulic action are similar. However, their construction is different. A slotted roller bucket has slots in its d/s circular portion – it thus has teeth or dentate surface on the downstream quadrant and a downstream apron that modifies the flow pattern and improves the flow conditions. Slotted roller is in general an improvement in design that reduces the possibility of extraneous material being drawn back into the roller, and due to its hydraulic behavior it acts as a self-cleaning structure. However, it is not preferred these days as its teeth are prone to suffer damages. Figure 5-24 shows both types of Roller Buckets, solid roller and slotted roller buckets, with the location of cylindrical rollers with horizontal axis.

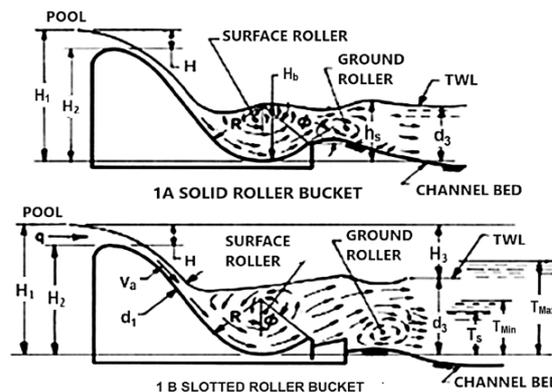


Figure 5-24: Roller bucket energy dissipators (IS 7365:2010)

### REFER TO APPENDIX B.5

Maneri Dam –Rehabilitation due to severe damage by abrasion

For hydraulic design and functioning of solid roller bucket and slotted roller bucket IS 7365 may be referred to.

### 5.3.3 Flip bucket

This type of bucket deflects the flow upwards and is not an energy dissipator by itself. The energy is dissipated by internal friction within the jet, interaction of the jet with the surrounding air, diffusion of the jet in the tail water and impact on the channel bed. This terminal structure is located at the downstream end of a spillway and is so shaped that the water flowing as a high velocity jet is deflected upwards in a trajectory away from the bucket. It is also called “trajectory bucket” or “ski jump bucket” (USACE, 1992).

An important aspect is the effect of erosion at the zone of impact of the jet, on close-by structures and, especially near the spillway dam. A RCC apron (anchored to rock) is these days generally provided adjacent to the flip bucket to take care of any scouring near the bucket due to low discharges as per IS 7365. In some new dams a preformed plunge pool has also been provided. Figure 5-25 shows a sketch of a standard flip buck-

et (free jet) with its geometry and exit flow parameters.

a) Basic hydraulic functioning

The parameters required to be evaluated in a flip bucket are:

- Hydraulic: depth (m) and velocity (m/s) of incoming flow at bucket invert and bucket lip, horizontal throw distance from bucket lip to the center point of impact with the tail water, the vertical distance of throw above the bucket lip level, and hydrodynamic pressure acting on boundaries (bottom and walls) for structural purposes.
- Geometry: radius of bucket, lip angle at the end of the bucket, depth of tail water below/above the bucket lip.
- Zone of discharge: tailwater level/depth (TW), type of material d/s of the bucket (geology).
- For hydraulic evaluation or design IS 7365 can be referred to.

According to USACE (1992), flip buckets “perform best when the entering flow is at high velocity and low unit discharge as such conditions result in considerable fraying of the jet by air resistance. Moderately high unit discharges, however, should not be a problem if downstream channel adjustment is not of prime consideration”. The use of flip bucket for high head and unit discharge (unit discharge over 130 m<sup>3</sup>/s/m), is a matter of discussion by the ICOLD.

The geometry of the flip bucket is related to the hydrodynamic pressure of the flow; in cases of increasing discharge, the adequacy of radius of bucket under a larger pressure should be evaluated, especially for short radius structures and also for possible operative condition with higher tail water i.e. when it is above the lip level. Usually the radius of curvature/bucket (R) is evaluated by expressions as:

- $R \geq Kd_1$  (K=5; Mason, 1993), (K=3, IS-7365-2010)
- $R/d_1 = 4Fr - 15$ , for  $5.5 < Fr < 10$  (Rao, 1982)

where  $d_1$  is the depth of flow (m) entering the bucket and Fr is the Froude number of the incoming flow. The total pressure at the bottom of bucket (hydrostatic + dynamic) is given by Mason (1993) as:

$$\frac{P}{d_1} = 1 + \frac{q_2}{g \cdot R \cdot d_1^2}$$

where: P = pressure head (m) and other parameters are as per Fig. 5-25.

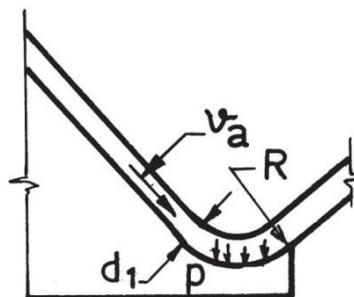


Figure 5-25: Sketch of a flip bucket (IS-7365)

The usual setting of flip bucket, for best performance, is when the take-off lip is above the tail water levels for all discharges; however, depending on river characteristics, cases with tail water levels greater than the bucket lip’s level are also frequently encountered and could also be adopted. Khatsuria (2010) and IS-7365 mentions that submerged flip bucket is an accepted practice in India’s spillways. Research in models indicates some hydraulic implications of the low bucket lip location as under (Khatsuria, 2005):

- Flow instability at bucket lip followed by reduction in pressure to sub-atmospheric with increase in submergence. Up to a certain depth of submergence, the pressures remain positive.
- Reduction in the throw of the trajectory.
- Reduction in the scour.

- Fall-back of the trajectory into the bucket and functioning as a submerged roller bucket for cases with very high submergence.

A particular hydraulic performance of the flip bucket (especially for ungated spillways) occurs during small discharges. The energy of flow is not enough to throw the jet and a hydraulic jump develops on the bucket, this condition is called “flow shocking”; in this case, water overflows the lip and falls on foundation close to the structure. According to type of foundation material, progressive erosion can endanger the stability of the structure. If needed, local protection measures are installed downstream of bucket. It is now a common practice in India to provide a RCC apron anchored to the rock adjacent to the flip bucket. (See IS-7365). Another design approach to solve flow shocking is by adopting a small radius for the bucket surface and a lower exit angle, and sometimes including a drainage’s cut at the lip.

Two aspects are key for the performance of the flip bucket:

- Jet trajectory
- Zone of jet impact at the river and consequent erosion process

b) Jet trajectory

The shape of the jet trajectory from the bucket lip to the point of impact with the tail water surface is given by a parabolic equation with origin at the lip’s edge, as presented in Figure 5-26. The best exit angle is less than 45 degrees, commonly about 30 to 35 degrees.

Trajectory Length

The following expression may be used for calculating the throw distance under (Khatsuria, 2005):

$$\frac{X}{H_v} = \sin 2\phi + 2 \cos \phi (\sin^2 \phi + Y/H_v)^{0.5}$$

where,

X = horizontal throw distance from bucket lip to the central point of impact with tail water (m);

Y= difference between the lip level and TW level, sign taken as positive for tail water below the lip level and negative for tail water level above the lip level (m);

H<sub>v</sub> = velocity head of jet at the bucket lip (m)

ϕ = bucket lip angle with the horizontal (°, degrees).

Vertical distance of throw above the lip level may be calculated from the following formula:

$$a = \frac{V_a^2}{2g} \sin^2 \phi$$

where,

a = vertical distance from the lip level to the highest point of the center of jet (m),

v<sub>a</sub>= actual velocity of flow entering the bucket (m/s),

ϕ = bucket lip angle with the horizontal (°, degrees)

g = acceleration due to gravity (m/s<sup>2</sup>)

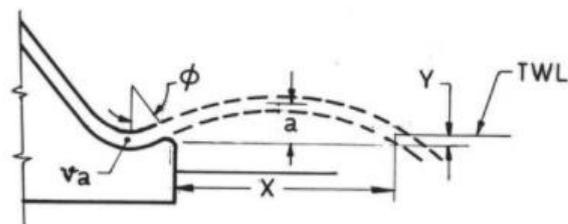


Figure 5-26: Jet’s trajectory (IS-7365)

In relation with the trajectory equation, actual throw distance could actually be lesser than that given by the projectile’s parabola due to air resistance on water jet (Figure 5-27), which depends on jet velocity at the lip; there are equations which include reduction factors, also some recent equations have been developed from tests on models. Figure 5-28 shows results of trajectory in laboratory tests for four flip bucket configura-

tions and variable discharge and water depth; (Configurations 4, 5, 6 and 7 correspond to design discharge with each bucket geometry, resulting in different jet trajectory). According to flow turbulence, the jet can “disintegrate” meaning the combined effects of dispersion and aeration. (See Figure 5-28)



Figure 5-27: Jet trajectory and effects of dispersion and aeration, Guri dam, Venezuela (Marcano, 2000)

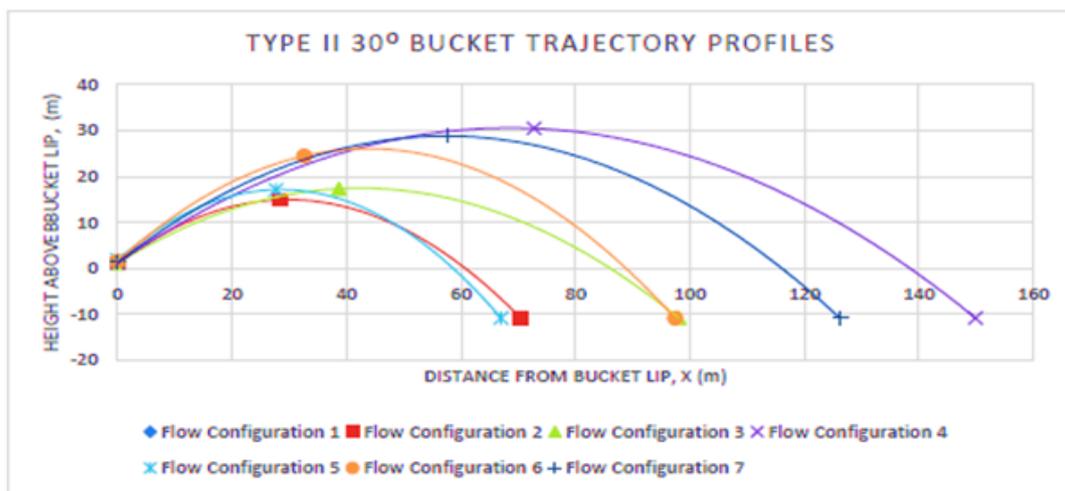


Figure 5-28: Flip bucket trajectory profiles for 30° lip angle (Neal Fraser C, 2016)

c) Zone of jet impact at the river and erosion process

The water jet impinges on the river bed with or without water cushion or plunge pool, generating several processes in the mass of water and on the ground such as turbulence, air entrainment, impact force, pressure fluctuation, drag and uplift of bed material. Scour is expected to happen in any type of material, even in hard rock; this process could be retrogressive and can move towards the bucket. In spillways, the site selected for jet impingement is almost always an adequate rock outcrop or rock strata with relatively shallow overburden. The basic need is that the resulting scour has to be limited not only in depth but also in extension towards other structures and river’s banks (or in the river valley slopes).

There are two aspects to highlight about the jet erosion process:

- In turbulent flow, shear stress is not the only cause of erosion. It is due to several other processes also such as turbulence, air entrainment and hydrodynamic pressure fluctuations.
- Rock compressive strength is not a unique parameter related with erosion (it is more significant in soft rocks), but is also dependent on various physical conditions of rock such as discontinuities, fractures, cracks pattern, blocks sizes, others.

For high tail water level, the impact zone becomes a plunge or stilling pool in which the discharge dissipates energy, reducing or avoiding bed degradation. In order to be effective, the pool depth has to be enough,

in relation to the energy and thickness of jet. This pool can be either natural (formed by erosion of riverbed) or artificially excavated as a part of the energy dissipation concept. According to bed material, this pool could be unlined and, in some cases, lined or improved with local protection (concrete) or with rock treatment-stabilization by way of anchoring of rock blocks, filling of cracks, grouting or dental concrete (filling of cavities or holes).

There are several approaches and models of rock scour based on hydraulic and geo-mechanical conditions. Figure 5-29 presents a mechanism of scour in rock (partially or totally fractured rock) due to a high velocity impinging jet, as proposed by Bollaert in his “Comprehensive Scour Model” (CSM). Scour is time-dependent, so it progresses according to magnitude and frequency of discharges up to a steady condition or ultimate scour. Figure 5-30 presents a plunge pool resulting from erosion on hard rock (granitic gneiss) during 15 years (showing monitored ground profiles in 5 years).

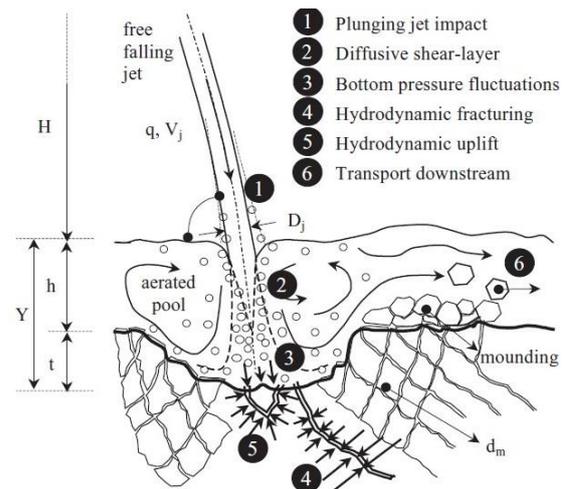


Figure 5-29: Main physical processes and parameters of scouring process, used for CSM (Bollaert and Schleiss, 2001)

REFER TO APPENDIX D

Hydraulic Modeling

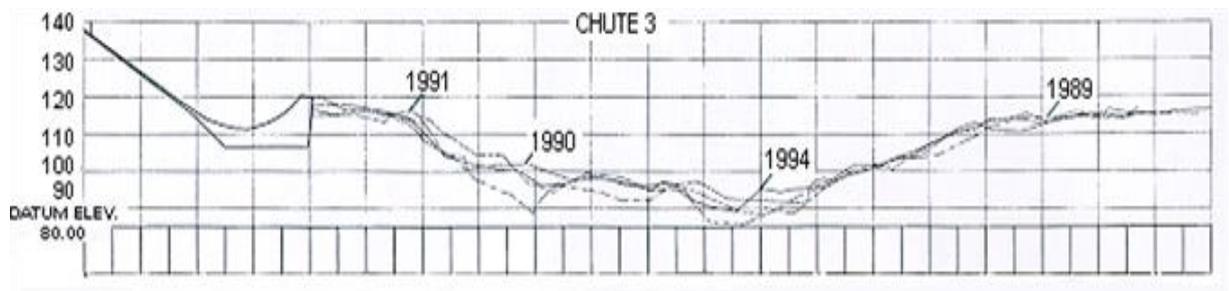


Figure 5-30: Formation of a plunge pool by jet erosion. Guri dam, (Venezuela) (Marcano, 2000)



Figure 5-31: Excavated plunge pool, Caruachi dam, Venezuela;  $q = 200 \text{ m}^3/\text{s}/\text{m}$  (Marcano, 2000)

Estimation of scour depth for evaluating spillway's safety is presented later in this chapter as "Erosion downstream of Terminal Structures". The scouring process can be reduced by inducing jet disintegration with ancillary elements at the lip, specific shapes of bucket and aeration. It is important to mention that scour could be avoided but with expensive structural measures; a better approach is allowing the water jet to develop a scour hole and a water pool. Since water's depth at pool depends on the impact of jet on the bed, sometimes a tailwater dam (tail pond dam) is located downstream to raise the tail water, but this option depends on the required depth of the pool. A preferred option is an excavated plunge pool to a defined distance downstream of deflector. Figure 5-31 is an example of excavated plunge pool for an IDF of 30,000 m<sup>3</sup>/s with a pool depth of 25 meters below tail water level.

d) Examples of bucket types energy dissipators in Indian dams:

These type of energy dissipators have been widely used in India. In a sample of 54 spillways with data from several references, including dams more than 50 years old, the energy head varies between 25 a 120 m. Figure 5-32 shows the distribution of use of bucket type energy dissipators. Figures 5-33 to 5-41 show examples of bucket type's energy dissipators in spillways of India's dams.

REFER TO APPENDIX B.1

Design and performance of Flip Buckets

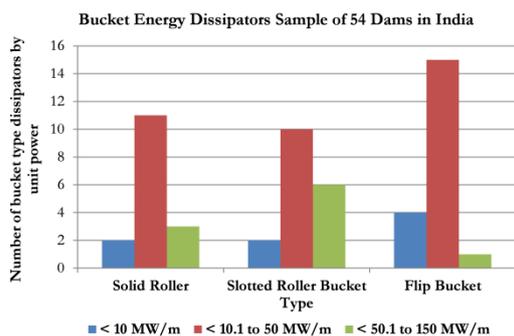


Figure 5-32: Bucket type energy dissipators in India's dams (several data sources)



Figure 5-33: Ichari dam, height 59 m, Slotted Roller Bucket, Q= 13,500 m<sup>3</sup>/s



Figure 5-34: Damanganga dam, height 58.6 m, Solid Roller Bucket, Q=22,040 m<sup>3</sup>/s



Figure 5-35: Gandhi Sagar dam, height 62 m, Flip Bucket



Figure 5-36: Dharoi dam, height 46 m, Solid Roller Bucket,  $Q = 21,662 \text{ m}^3/\text{s}$



Figure 5-37: Maithon dam, height 50 m, Flip Bucket



Figure 5-38: Kadana dam, height 66 m, Solid Roller Bucket,  $Q = 49,497 \text{ m}^3/\text{s}$



Figure 5-39: Ranapratap Sagar dam, height 53.8 m, Flip Bucket



Figure 5-40: Ukai dam, height 105 m, Flip Bucket,  $Q = 42,269 \text{ m}^3/\text{s}$



Figure 5-41: Madhuban dam, Flip Bucket

### 5.3.4 Energy dissipaters for orifice and bottom spillways

Chapter 3 describes Orifice spillways and Bottom Outlets. These type of appurtenance are commonly used at run-off river dams in mountain rivers. Three of their operative conditions are fundamental to cope with energy dissipation:

- Discharge of sediments: flushing of sediments with sizes up to boulders, logs and other debris.

- Profile: the crest or sill is set at low elevation or at the bottom of reservoir, which controls its profile from the crest of inlet to the terminal structure.
- Flow: usually they release high discharge per vane (controlled, commonly, by radial gate).

The design and operation of energy dissipaters under these conditions is particularly complex, hydraulic performance and structural durability is always a challenge.

According to project characteristics and river morphology, two types of energy dissipators are used, and commonly found in those mountain dams: Flip Bucket and Stilling Basin (adapted to local conditions). There is no standard procedure for these special dissipators. Several researchers, in particular experience of India (Khatsuria, CWPRS and others), present several aspects related to hydraulic performance of dissipators in these operative conditions.

The convenient option is a flip bucket, whose profile facilitates the pass of heavy load of sediments and guarantees a direct free discharge to downstream channel; however, in some installations, submergence of deflector is a matter of concern. Even though stilling basin can be used, its hydraulic efficiency could be affected by the flow concentration, the low profile and the amount of sediment; thus, basins are long and not so deep structures where the hydraulic jump could be unstable, with difficulty to have an adequate tailwater in the channel. In both cases, sediment accumulation and flow obstruction, in the structure or in downstream channel, is an key aspect for hydraulic performance of these energy dissipator.

The typical scheme of this bottom spillways encompasses the sluice with the inlet structure (a rectangular orifice with shaped edges and the gate; see Chapter 3) followed by a low (flat) slope glacis/chute and the energy dissipator. For flip bucket, and according to river profile, the lip of the deflector can be located at a higher level than tailwater so it functions with free discharge or to a lower level, with partial submergence. This scheme is frequent, and a dual style of operation could be expected as deflector and as a basin with drowned hydraulic jump, according to discharge and tailwater. Figure 5-42 shows a case of orifice spillway with combined operation.

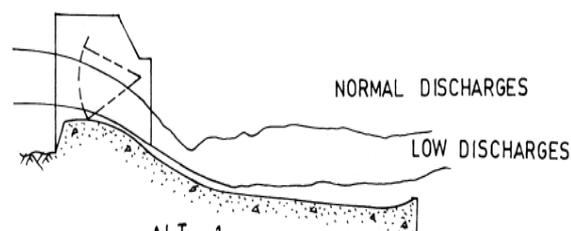


Figure 5-42: Bottom outlet, combination of energy dissipation (Khatsuria, 2005).

Some factors to be considered in evaluation of existing energy dissipators of orifice and bottom spillways: that can be causes of poor performance, are (Castro,C.; Khatsuria, R.M.; CWPRC and others institutes):

- Length of structure: since flow concentration is high, the dissipation structures tend to be long. This is especially important in stilling basins where the common option is an open section without ancillary elements, so hydraulic jump develops its entire length. Short basins favor inadequate performance due to instability of jump, either it is drowned or washed away.
- Sediment, amount and type: the sediment load should be passed down with minimal accumulation and obstruction of structure. The profile of the invert is fundamental for this cleaning and flushing action; deflectors with high lips and deep basins are prone to deposition of solids. Profile as that shown in Figure 5-42 favors a better performance.
- Natural channel: the characteristics of the channel (geology, profile and confinement) not only control the profile of the spillway but also define basic conditions for hydraulic performance of the dissipator, as: tailwater for different discharges (free or submerged operation), potential erosion process at the channel according to physical condition of local material (risk to undermining of the structure or banks instability) and potential obstruction with sediments (added effects on tailwater).

- **Flow concentration:** as mentioned, commonly, these bottom outlets release high discharges even for relative small area of orifice, so there is a high flow concentration and velocity, this high energy flow can modify the energy dissipation for different discharges, for higher updated IDF or even lower than IDF.
- **Abrasion:** high velocity sediment-laden flow (coarse sediment as sand, gravel, boulders), abrasion is an important detrimental factor for the entire spillway due to cumulative damages to concrete and its effects in stability of the structures and durability. Ancillary elements are particularly prone to abrasion. Use of special concrete cover, steel lining or steel elements, are some options to protect the spillway.
- **Cavitation:** high velocity flow on flat profile or passing on damaged concrete surface (by abrasion) can lead to cavitation processes. It can be controlled by aeration; however, in flat slabs could be difficult to define an aeration section, especially if the chute-dissipator are partially submerged. The irregularities of concrete surface can trigger cavitation processes. Ancillary elements are particularly prone to cavitation.
- **Flow distribution:** in some cases flow along the glacis and energy dissipator is not evenly distributed across the section due to factors as: location of inlet, river morphology at the approach zone, hydraulic functioning of inlet and gate, sediment accumulation, gate operation, and others. This condition affects energy dissipation and favors the occurrence of local and intense deleterious processes: shock waves, abrasion, cavitation.
- **Other hydraulic loads:** adding high velocity flow and submergence can lead to similar hydraulic actions than those in chute and energy dissipators in conventional spillways as: hydrodynamic pressure fluctuations, uplift variations, vibra-

tion, high turbulence and roller action with capture of coarse material from riverbed into the dissipater. All these loads with similar adverse responses of the structures.

In India, in the Himalayan region, there are several dams with orifice/bottom spillway, functioning under a heavy load of sediments and rolling boulders. The main reported damages includes abrasion and impact of rock. Among those dams, within the DRIP rehabilitation program, are: Maneri, Ichari and Uttarakhand dams. Figure 5-43 shows Chamera II dam.



Figure 5-43: Bottom outlet spillway. Chamera II Dam, India (Bhajantri et al, CWPRS, Pune)

### 5.3.5 Bucket type energy dissipaters for outlet works and tunnel spillways

The bucket type energy dissipators have been used as terminal structure of tunnels and sluices also in addition to stilling basins. Among the bucket type of energy dissipaters the most common type used has been the flip bucket since the tail water requirement (submergence) for roller buckets is normally difficult to satisfy in outlet works. In India the use of large tunnels for main or auxiliary spillway is not frequent, but sluices are very common in concrete dams and integrated to surface spillway. Out of 30 large dams reported by ICOLD (2016) with tunnel spillways, only one is located in India i.e. Tehri Dam. The spillway arrangement includes

four tunnels (2 no. each on both banks) for  $1,850 \text{ m}^3/\text{s}$  each and a total head of 200 m. (see Figure 5-44). The tunnels, and surface spillway, discharge into Koteshwar dam's reservoir, both dams together constitute the Tehri hydropower complex.



Figure 5-44: Tehri dam, India, surface spillway and outlet of right bank tunnels.

The geometry of flip bucket is adapted to the flow conditions/geometry of the tunnel, usually the structures are short and shaped to avoid backward effect; the emerging jet is discharged into an adapted channel. As for spillways, the safe distance of impingement have to be defined for the range of discharges in order to avoid damages on account of erosion near the bucket and other near structures. The flip buckets could be solid or with dentate lip to favor the energy dissipation downstream. The plunge pool can be natural, excavated in rock or protected with concrete. For natural pool the ultimate depth of scour is estimated as for spillways. Figures 5-45 shows a typical geometry of flip bucket for a tunnel outlet.



Figure 5-47: Three Gorges dam, China, bottom outlets (sluices), exit velocity of  $35 \text{ m/s}$ , (Chanson, 2015)

Figure 5-47 presents the terminal structure for the bottom outlets of Three Gorges dam, China. Figure 5-46 shows an example of slotted flip bucket and Figure 5-48 shows a special scheme of the energy dissipation (by whirl motion in horizontal tunnel which is located at an eccentricity with respect to vertical shaft) for right bank tunnel (Morning Glory) spillway, for velocity of  $55 \text{ m/s}$ , in Tehri Dam.

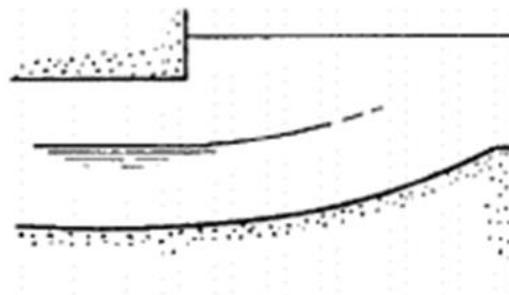


Figure 5-45: Typical geometry of flip bucket at tunnel outlet (USBR, 2014)

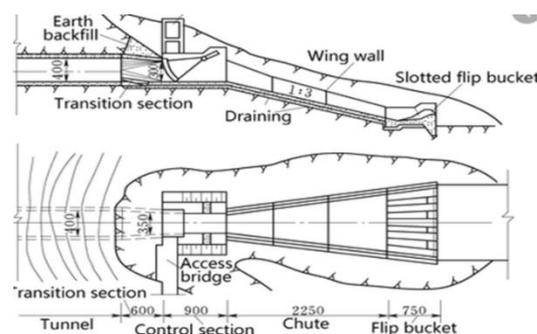


Figure 5-46: Layout of terminal structure in a tunnel with slotted flip bucket (Chen, SH, 2015)



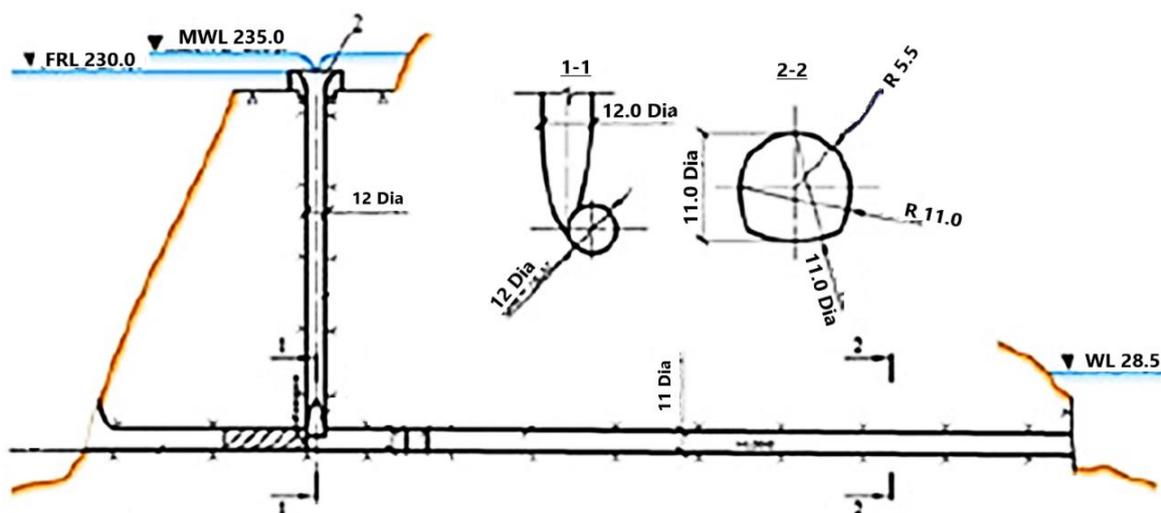


Figure 5-48: Tehri dam, India, Morning Glory spillway, swirling type of energy dissipation arrangement.

### 5.3.6 Other types of energy dissipation for spillways and outlet works

#### 5.3.5.1 For spillways

Other forms of energy dissipation in spillways are:

- Energy dissipation at the toe of dam by water falling over tail pool on riverbed (natural or protected with concrete slab). This is a common option in concrete arch dams (See Figure 5-49).
- Dissipation along the chute with baffle slab for low dams or steeped chute as typical solution in Gravity Rolled Compacted Concrete (RCC) dams (See Fig-



Figure 5-49: Free jet with energy dissipation at toe of Arch dam.

ure 5-50, and explanation of functioning in Chapter 3 of this Manual).

- Special energy dissipators tested in hydraulic physical model. Some of them might be variations of standard designs of stilling basins or bucket types (See Figure 5-52).
- Combined functions of one energy dissipator: Surface spillway plus sluices (See Figure 5-51).

#### 5.3.5.2 For outlet works

Others types of energy dissipators for outlet works are:

- Impact basin
- Energy dissipating valves



Figure 5-50: Stepped chute on RCC dam



Figure 5-51: Combined Flip Bucket. Hirakud dam (India), surface spillway and sluices

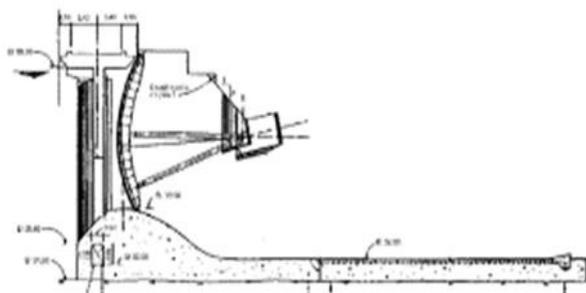


Figure 5-52: Horizontal apron, dissipation on rock's riverbed; Macagua dam, Venezuela (Marcano, 2000)

a) Impact type:

This structure dissipates kinetic energy by striking of the jet emerging from the conduit into a vertical baffle located in a chamber and then by turbulence. The main advantage of this energy dissipator is in its independence of tailwater (TW) and its compact design. There is a standard structure named as impact basin USBR Type VI (USA) recommended for a maximum incoming flow velocity of 15 m/s, Froude number less than 10 and maximum discharge of about 11 m<sup>3</sup>/s. For larger discharges, multiple basins could be placed side by side. Figure 5-53 shows a typical double impact basin. The

REFER TO APPENDIX B.2

Energy Dissipation at Macagua Spillway

fundamental dimension of this basin (or chamber) is its internal width (W) which is related to discharge, other dimensions are proportional to that width. To define W, USBR proposed as under:

$$W = K * Q^{0.4}$$

where,

Q = Discharge (m<sup>3</sup>/s)

W = Internal width of chamber (m)

K = Factor. (K= 1.864 for lower limit of W and K= 2.27 for upper limit of W.)



Figure 5-53: Impact basins (USBR Type VI), two structures placed side by side (FEMA, 2010)

b) Valves as Energy Dissipators valves:

These are valves with free discharge to atmosphere with dual function of discharge's control and energy dissipation:

- Fixed cone valve also named as Howell Bunger valve (after its developers at USBR)

- Sleeve valve
- Hollow Jet

In this part of the manual, only the Fixed Cone (FCV) valve is presented since it is currently the most used as terminal device for outlet works. Hollow Jet valve was presented previously associated to its need of a stilling basin energy dissipator; it can be found in installations in medium age dams.

FCV provides controlled discharge of water while protecting the downstream environment. The valve breaks up the water into a large, hollow, expanding spray and can be used in most situations, including submerged applications. They are ideally suited for outlet works for power projects, flood control systems, irrigation facilities, and draining reservoirs or ponds.

The size of the valve is selected from the manufacturer curve and determined by the maximum available net head at the valve. Net head is the distance between the head water elevation and the centerline of the valve or if the valve is submerged, the tail water elevation less the upstream pipeline head losses caused by inlet, conduit, reducer, bend, etc.

The FCV valve discharge is an expanded cone of water, at a 45° angle (approximate), with energy dissipation by dispersion in air. This discharge avoids erosion of surrounding area. In some cases, if wide water dispersion (spray pattern) is objectionable (environmental impact) or subject to freezing or ice build-up, a hood can be added to create a



Figure 5-55: Fixed cone valve (Howell-Bunger) and typical discharge (diverged and hooded)

jet-like stream; and for high head, a concrete confinement structure or a vertical stilling well, with due ventilation, has been used with good jet performance. The highlighting features of this FC valve are: ease of operation, high flow handling with excellent hydraulic performance, cavitation-free and durability.

The discharge curve of the FCV is:

$$Q = Cd . A . (2g . H)^{1/2}$$

where:

Q = discharge (m<sup>3</sup>/s)

Cd = discharge's coefficient which varies with stroke (according to manufacturer's curve, Cd = 0.82 to 0.86, for full open)

A = area of inlet pipe (m<sup>2</sup>)

H = net head at inlet of valve (m)

This valve can be installed for heads up to 150 m. For reservoir installations its size can be up to a diameter of 3.35 m. Figure 5-54 shows a hooded FCV discharging jet directly into a plunge pool and Figure 5-55 shows a Fixed Cone valve.



Figure 5-54: Fixed cone valve with hood and concentrated jet into a plunge pool (FEMA, 2010)



## 5.4 Description of Exit Channel and Plunge Pool

The terminal structures encompass the component that dissipates the flow energy and the channel that conveys the flow to the river.

### 5.4.1 Exit channel

For various types of energy dissipators a conveyance work is required that finally takes the flow to the main channel or river course; usually this exit channel is required for:

- Stilling basins of spillways and outlet works
- Roller and flip buckets of spillways and outlet works
- Specific energy dissipators in outlet works

In stilling basins and roller buckets, especially in cases of flank spillway (not in the river bed), there is usually a transition zone before the flow enters in to the main channel/river course, which can be a widened channel stretch which helps to dissipate the remaining energy (not dissipated in the basin) by the formation of eddies and turbulence. This is called exit channel or spill channel. Depending on the type and strength of the natural material encountered in this transition zone(exit channel), protection works may be required to avoid its bed and banks erosion

The most common types of protection are: Rip-Rap, Gabions, Articulated Concrete Blocks (ACB) and Geotechnical products with or without vegetation. Figures 5-56 to 5-59 present several types of bank protection in channels. In existing exit channels, these types of bank protection, among others, can be used to rehabilitate eroded stretches considering availability of materials, cost, site characteristics and ease to construct.

As mentioned, the exit channel sets the tailwater level (TW). This level is defined, naturally, by the backwater profile in the channel, which, for subcritical regime will tend to the normal depth at its upstream end, close to the basin. In other cases, a downstream control component (weir) is required to raise the water level and reach the required TW. In case of increased discharge through the spillway, the Channel Discharge Curve or Tail water rating Curve, in the section immediately downstream of the basin, allows to define the TW and to verify the behavior of the energy dissipator. Since in stilling basins it is common that bed level of exit channel in case of flank spillway or river channel in case of central spillway is higher (as the case may be), a smooth upward slope is required for connection; in the case of roller buckets the effect of turbulence in this counter-slope stretch, can move loose material into the bucket resulting in serious abrasion damage, so bucket invert should be carefully established.



Figure 5-56: Geotechnical product with vegetation, bank protection in a natural channel



Figure 5-57: Articulated concrete blocks (ACB) as mattresses, bank protection in excavated channel



Figure 5-58: Gabions, Reno mattresses, bank protection in excavated channel



Figure 5-59: Rip-Rap, bank protection in natural channel

### 5.4.2 Plunge pool and river environment

In flip buckets, the transition zone downstream the energy dissipator is a plunge pool joining to either the exit channel or to the riverbed. Previously the erosion process in the impingement zone of the jet was presented, which can be reduced with the creation of pool of adequate depth; this pool can be natural or created by excavation or with a dam pond downstream (or combination of both measures). If the pool is made naturally by erosion of the jet, several morphological river's responses can occur, depending on type of bed material:

- Bar formation of moved material. This accumulation of sediment and debris bars can obstruct the channel, can modify the TWL and affect the performance of the energy dissipator.
- Meandering of the riverbed in wide valleys.
- Abrupt widening or narrowing of river bed.

Another important aspect in the plunge pool is occurrence of waves, surges and strong turbulence, which can be transmitted for several hundred meters along the exit/river channel and can generate instability of valley slopes and bank erosion. Figures 5-60 to 5-62 show examples of terminal structures and exit channels. In many cases, for design and evaluation of plunge pools and exit channels, physical models are required;

in the last decade, CFD models have become a common tool to evaluate flow's pattern and its effects, as shown in Figure 5-63.



Figure 5-60: Flow pattern at exit channel in a low head spillway



Figure 5-61: Flip bucket, plunge pool and exit channel in a high head spillway; Guri dam, Venezuela (EDELCA, 2000)

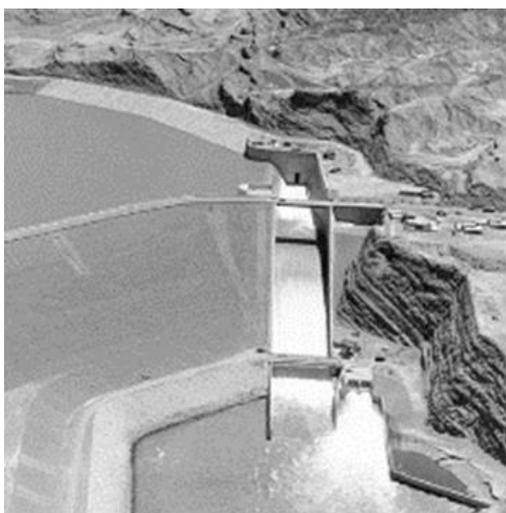


Figure 5-62: Combined exit channel for spillway and outlet work; Ullum dam, Argentina (Harza Eng. Co.)

REFER TO APPENDIX B1, B2, B3 and B4

For plunge Pool

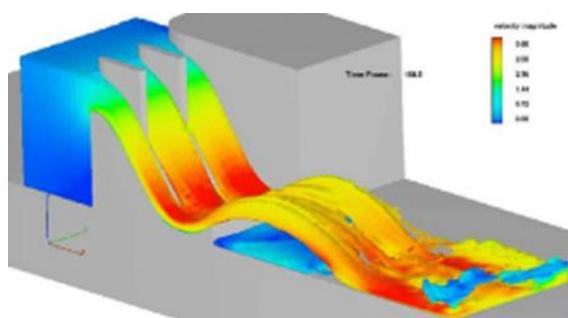


Figure 5-63: Flow over spillway, flip bucket, plunge pool and exit channel, 3-D, CFD Model-velocity

REFER TO APPENDIX D

Hydraulic Modeling

## 5.5 Assessment of Hydraulic Safety of Existing Terminal Structures

This paragraph integrates the types and characteristics of terminal structures (for spillways and outlet works) with the hydraulic

hazards (or loads) that may arise during their operation for different discharges ranging from frequent flood to design flood (or hydrological updated IDF).

The evaluation of the physical conditions of the terminal structures and their hydraulic operation is an activity prior to the rehabilitation of their components. This evaluation brings together field activity such as the detailed inspection by experienced engineers and technicians and office activity, which includes hydraulic calculation for checking performance for the expected discharges. In some cases, due to type of hydraulic structure or by flow conditions, the use of mathematical models or tests in physical models could be required.

The need for rehabilitation is to be preceded by the evaluation of existing terminal structures for the following two cases:

- Hydraulic malfunctioning for the original design discharge or even for lower discharges. This becomes evident when the structural elements present physical damages.
- Upgrading of spillway. All the components must be checked and adapted for higher discharge according to hydrological update of the design flood (IDF). This is the case when the updated IDF is greater than the original design flood.

### 5.5.1 Evaluation of hydraulic safety of terminal structures

The participation of the terminal structures in the safety of the spillway and, by extension, of the dam has to do with the proper management of the specific flow ( $m^3/s/m$ ) discharged i.e. intensity of discharge and its energy content. The flow rate corresponds to that used for the design of the spillway or resulting from design flood update (new IDF). As in other spillway's components, reliability is related with non-accomplishment of its hydraulic function during its working life; thus it covers both

serviceability and durability, under different situations: usual (frequent discharges) and unusual (IDF or rare flood events).

This manual refers to existing dams, spillways and its terminal structures which have a history of operation; they may need to be rehabilitated for both the original design flood as well as the updated IDF. As mentioned in Chapter 3, for a spillway, hazards come from its exposure to floods (IDF or other floods) and to various flow conditions, the first is the typical hydrological scenario and the second responds to hydraulic causes.

In a spillway's terminal structure, the basic hydraulic function has to do with:

- Ability to cope with high energy content flow and to deliver (or to return) the flow to the river without damage to any structure and the environment of the riverbed.

If this requirement is not complied with then there is a possibility of potential failure of the spillway and a hazard for the dam-reservoir system with serious consequences to the owner not only for the cost involved in its reparation but also for the potential hazard to other structures of the reservoir.

This chapter covers assessment of hydraulic safety of terminal structures and exit channel, defined as their response to the occurrence of any of the following loads conditions:

- Due to discharge capacity :
  - IDF (“as originally designed”)
  - Flood greater than original IDF (updated IDF)
  - Frequent floods (any discharge lower than IDF)
- Due to flow

- Any hydraulic action due to flow regime with high velocity (high energy) for any discharge.
- Erosion downstream of terminal structures.

The condition imposed by a flood greater than original IDF in an existing dam is the worst scenario of hydraulic safety (or the critical failure mode) because of high risk of damage or malfunctioning, or even potential overtopping with eventual dam break. If the increased discharge can be handled by the spillway, all components of the dam must also be checked to see whether they are able to cope with this discharge and associated flow conditions. In certain cases, increasing spill capacity requires provision of additional spillway or upgrading of the control structure with a non-conventional weir, to reduce dam hazard. In any case, the performance of terminal structure must be checked since it could become a limitation for this overload. For occurrence of IDF, the hydraulic capacity of the spillway, conveyance structure and energy dissipation should be enough and safe; so evaluation of an existing spillway, its performance and rehabilitation should focus on this criteria. For high frequency floods, the hydraulic loads can develop progressive and accumulative damage to spillway's components, in this case, to terminal structures and exit channel.

The condition of high energy flow introduces several scenarios of loads where the hydraulic safety in the terminal structure (energy dissipator) and exit channel may become affected. The following paragraph taken from chapter 3, applies in its entire text to these components also: “For these load conditions all modes of malfunctioning or failure are triggered by hydraulic actions, but adverse response is expected in a structural element, so rehabilitation's measures are mainly structural. Other aspects related with this hydraulic functioning have to do with frequency, duration and repetition of the loads acting on the elements. This means that incidents not only happen in one

large event of flood but also could occur by accumulating effects or damages from many frequent events (much lower than IDF) as “progressive failure” during operational life, then, suddenly, a structural element fails or breaks”.

In spillways, a failure mode other than malfunctioning of the control structure, is commonly defined as “non-critical” because they, usually, do not produce incidents that endanger the dam. However, in the case of terminal structures and exit channel, failures modes associated with deep erosion could become critical due to danger to the spillway and in extreme cases, endangering the dam directly. There are examples of back erosion from exit channel with failure of the whole spillway and other structures and serious damages to the embankment dams. Richards et alia (2017) “highlight the importance of evaluating and understanding the spillways exit flow conditions in dams”, presenting several recent cases of serious incidents (with discharges lesser than design) in embankments dams in USA and proposing to include plausible failure’s modes of the dam due to exit flow from spillways.

As mentioned in Chapter 3, “it is important to mention that surveillance and maintenance of spillways are key activities for manage the risk due to hydraulic actions. In this case surveillance includes the ability to detect deficiencies or hidden damages and the effects of its progress to a future problem”.

The rehabilitation activities related to assessment of hydraulic safety and possible adverse responses of terminal structures and exit channels, have to do with:

- Limitation of discharging capacity of the spillway.
- Physical conditions of terminal structure and their effects on high velocity and turbulent flow.

- Tailwater impact on energy dissipator performance.
- Geological conditions of river bed and erosion potential.
- Human resources related activities. This chapter only mentions these topics, since details are included in the following CWC’s publications: “Guidelines for preparing Operation and Maintenance Manuals for Dams” and “Guidelines for Safety Inspection of Dams”.

Evaluation of hydraulic safety, as mentioned earlier, has to consider all possible adverse behavior of terminal structures and exit channel due to hydraulic actions. The response to those causes (loads) is understood as lack of robustness of the structure (due to dimensions, geometry, levels or stability) or existence of fragile elements which become critical for a satisfactory hydraulic performance (structural details, concrete surface, materials, joints). The integration of those hydraulic causes and structural responses leads us to possible modes of malfunctioning or failure of the component; in extreme conditions these events can become modes of failure for the spillway and even of the dam.

REFER TO APPENDIX A

Failure Modes

### **5.5.2 Inadequate capacity of the spillway**

Once updated IDF, a key task is to evaluate the hydraulic performance of terminal structure and exit channel. In some cases, evaluation of capacity of terminal structures has to be done by using mathematical or physical models. Increasing the discharge may change some or all the hydraulic actions and its effects as mentioned later in this paragraph.

The aspects related to geometry of terminal structure vary according to type of energy dissipator, these are:

- In stilling basin:
  - Length of the basin.
  - Use of ancillary elements on the apron: chute blocks, baffle blocks, impact or friction blocks and end sill.
  - Level of bottom (apron) of basin
  - Height of walls
- In roller types:
  - Radius of curvature of roller surface
  - Level of end lip
  - Use of slotted or dentate surface
  - Height of walls

The dimensions of stilling basin, its invert level and dimensions of its ancillary elements are related to the efficiency of performance by keeping the hydraulic jump within the basin. If the flood discharge increases then the hydraulic performance of the basin will not only be different but may be inadequate or hazardous if the jump is not controlled. In an existing stilling basin, for a greater discharge and new flow condition at the inlet of the basin ( $V$ ,  $F$ ), the efficiency of energy dissipation can vary due to:

- Required length of basin could be larger
- Tail Water depths may not match with the conjugate depths for higher discharges, stability of jump may not be guaranteed; it could result in drowned jump or sweep out.
- Dimensions and location of chute and baffle blocks and end sill, may not be adequate for the Froude number of the flow at inlet.
- Existing height of walls may not guarantee the required freeboard to avoid overflow.

- Flow pattern over end sill of basin could generate local erosion and undermining of the basin.

In the scenario of a sweep out of hydraulic jump, severe erosion process can occur around the basin (backfill and downstream) which can undermine the structure and produce its failure. In this case, the spillway loses its function of protecting the dam. Depending on the quality of the foundation material this erosion could occur in one high flood event or more. In the extreme case, back erosion and head cutting can damage the spillway completely. Figure 5-64 shows a spillway failure due to combined causes, mainly due to overtopping of walls, hydraulic jump swept out from basin and head cutting.

REFER TO APPENDIX B.1 and B.2

### Failure of Energy Dissipators



Figure 5-64: Hydraulic jump's swept out, head cutting erosion, loss of abutment with uncontrolled water release from reservoir, El Guapo dam, Venezuela

For roller buckets, their invert and geometry is associated with submergence due to tail water; so an increase in discharge could modify the flow pattern of the rollers and in

the downstream; these changes could increase local erosion and dragging of bed material into the roller. For flip buckets changes in hydraulic performance of the deflector could be less critical, if stability of the concrete bucket is not affected; however, for the geometry of the bucket and new flow conditions ( $V$ ,  $d$ ), the trajectory of the jet will change and also the impingement site, which could be a matter of concern due changes in the erosion patterns in the pool.

In relation to the tailwater, as this level increases with discharge also the induced uplift on the terminal structure increases for the scenario of operation during design flood; as shown in Figure 5-65, this condition of hydrostatic uplift should be checked for updated discharge (see IS-11527-R2004).

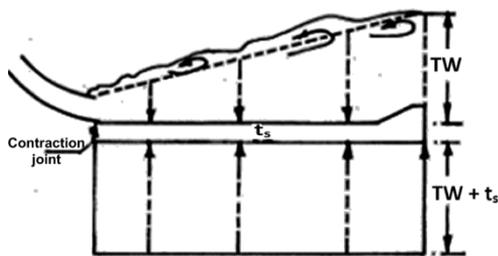


Figure 5-65: Hydrostatic uplift acting on apron of stilling basin (Khatsuria, 2005)

### 5.5.3 Condition of concrete surface of terminal structure

Similar to the conveyance features, occurrence of hydraulic actions are aggravated with changes in geometry and irregularities on surface of concrete. The adverse response of terminal structures to the different hydraulic “loads” may be local or extended to the whole structure, and could be sudden or progressive during a period until a major damage occurs.

These hydraulic loads are present for any discharge; even for those smaller than IDF.

The surfaces in energy dissipators may have physical irregularities resulting from:

- Displacement of elements or misalignment of boundary.

- Roughness,
- Joints: inadequate design, opening without any seal, damaged seal.
- Aging and degradation of concrete.
- Construction deficiencies in meeting surface offset tolerances.

As mentioned in Chapter 3 and in “Manual on Assessing Structural Safety of Dams” in terminal structures as in chutes, the effect of hydraulic loads is cumulative since deterioration on surfaces cause the flow effect to intensify, creating a cycle of progressive damage to the concrete element. The damages and eventual failure of concrete elements due to high velocity flow encompasses two processes: erosion (abrasion) and effects due to changes in local pressure (cavitation, vibration and hydrodynamic pressure fluctuations).

Since some of these processes are similar to those occurring in chutes, description will be summarized highlighting those specific aspects that apply to terminal structures. The reader is referred to Chapter 3.

In energy dissipators, the structure is exposed to an environment characterized by three fundamental aspects related with the generation of hydraulic loads:

- a) Flow: High velocity with changes of direction and in flow area (contracting and expanding sections), turbulence (or macro-turbulence), separation zones, vortexes, fluctuating pressures with singularities. In some cases, especially in controlled spillways, asymmetrical flow across the structure.
- b) Contour and obstacles: Presence of ancillary elements (different shapes and locations) that modify local flow conditions and favor occurrence of singularities, as: chute blocks, baffle (friction) blocks, end sill and flow splitters (slots and dentate edges in the buckets).

- c) Condition of concrete surfaces in existing structures. Detailed inspection of the work makes it possible to know if there are any surface damages (damages at early stage or clearly established), sediment accumulation, loss of structural concrete or other signs indicating that a hydraulic load is acting or a hydraulic phenomenon can be detonated.

Previous inspection of the spillway components and surrounding site is a key activity in the rehabilitation process; especially, to establish a “baseline condition” of the component. In some energy dissipators this task is difficult since they may be filled with water so access for an inspection of the structure could be a limitation. In some cases it is required to make use of specialized diver inspector, in other, remote operating equipment (ROV) allow to capture videos or photos of concrete surfaces and blocks. A recent technology based on the use a 3D scanner and data processing with specific software allows to define topography of concrete surface underwater, distribution of damages and depth/volume of the process (Hasan et al, 2019). Dewatering the basin is a feasible option but coffer dams are needed to close it, also uplift acting on the empty structure requires a detailed checking.

- Abrasion

The incoming high-velocity turbulent flow from the chute produces a shear force on concrete surfaces (apron, invert or walls); if there is suspended sediment load in the flow (silt, sand), also bed load such as gravel or rock fragments, and dragged material: de-

bris, ice or others, enter to the basin or bucket, abrasive capacity of flow can produce serious damages to the concrete elements, even break and loss of continuity of structural element. As in chutes, but very often with greater damages due to strong turbulence, the abrasion mechanism (friction + rubbing) can disintegrate the concrete and release fragments (ball milling action); the damage pattern is erratic and nearly parallel to the surface resulting in a rough surface. The process itself feeds abrasive material, so higher sediments concentration and fragments of concrete add to intensify damage at next discharge event. Abrasion intensifies in ancillary elements (blocks, slots and dentate) if their location expose them to high velocity flow. Hard fragments can get into the dissipators due to instability of local slopes, thrown by people or by being dragged from the exit channel due to roller action, return flows or vortexes and turbulence at the outlet. Figure 5-66 shows the recirculation of dragged fragments into a basin, this could be more intense in roller buckets due to eddy activity at outlet.

The intensity of the process depends on the frequency of passage of abrasive water and the quality/durability of the concrete, especially in old dams. The resulting uneven and rough surface increases abrasion and facilitates the occurrence of the other process i.e. cavitation. In extreme cases where baffle blocks are lost by abrasion, hydraulic jump can sweep out of the stilling basin adding its effect to erosion damages downstream. Figures 5-67 to 5-70 show stages of abrasion up to loss of structural element.

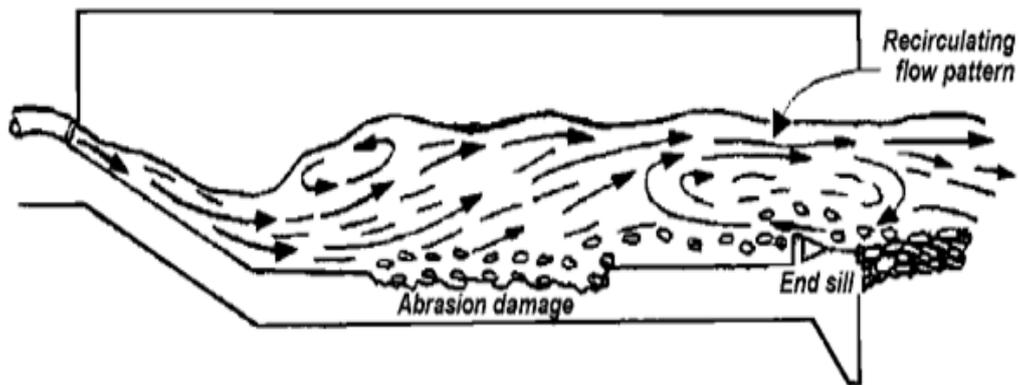


Figure 5-66: Recirculating flow pattern and rock fragments intrusion (FEMA, 2010)



Figure 5-67: Incipient abrasion damage of concrete surface (FEMA, 2010)

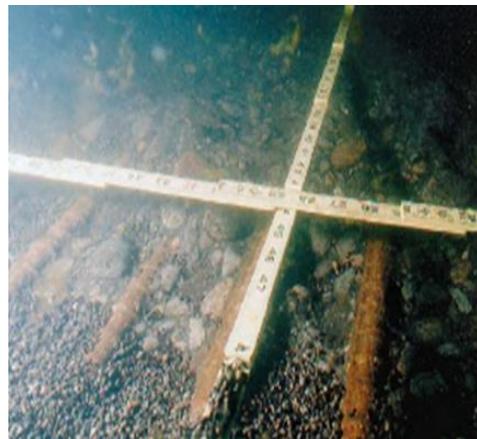


Figure 5-68: Advanced abrasion damage of concrete slab of a stilling basin (FEMA, 2010)



Figure 5-69: Severe damage by abrasion, bottom slab of stilling basin, Libby dam (USACE, 1980)



Figure 5-70: Severe damage due to abrasion, loss of baffle blocks (FEMA, 2010)

There are many examples of spillways with major damages due to abrasion in terminal structures. In India, several cases have been reported, especially in bucket type (solid and slotted) not only due to solid fragments dragged from downstream into the bucket due to roller action but also due to flow transport of gravel and boulder in low-medium head orifice spillways. Some roller bucket with major abrasion damages, presented by Khatsuria (2015), are: Pench, Barna, Ujjani, JawaharSagar, Bargi, Hasdeo-Bango, Damanganga, Dharoi, Kadana and Hiran II. A region with severe cases of concrete abrasion is “The Himalayas”, due to heavy bed load of gravel and boulders, as in dams: Bhakra, Ichari and Maneri (For damages in Maneri Dam see Figure 5-71).



Figure 5-71: Severe damage by abrasion in terminal structure, Maneri dam, India (CWC)

**REFER TO APPENDIX B.5**

**Damage by Abrasion in energy dissipaters of Maneri Dam**

An important example of abrasion damage and engineering approach for repairing the stilling basin is Bhakra dam, in Himachal Pradesh. This is a concrete gravity dam, 225 m high, with a gated surface spillway and a

stilling basin divided into two bays by a wall. The basin is 128 m long and 25 m deep, with a bottom slab 6 to 12 m thick. As described by CWC: “Bhakra Dam spillway apron floor has been damaged due to abrasion caused by churning action of boulders, concrete lumps and other metallic pieces that may have been sucked into the stilling basin from the downstream river bed or those which might have fallen in the stilling basin pond during the construction stage of the project. The extent of damage was found up to 16 inches (0.41 m) in depth”. Figures 5-72 to 5-74 show the dam and spillway located in between two major hydropower plants, and aspects about the abrasion damage at the bottom slab of stilling basin.



Figure 5-72: Bhakra dam, spillway and energy dissipator,  $Q = 5,587 \text{ m}^3/\text{s}$

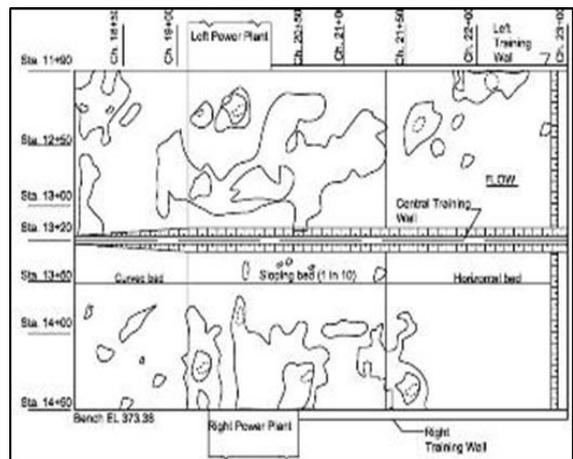


Figure 5-73: Bhakra dam, basin’s slab with topography of abrasion damages (from 0.15 to 0.41 m deep).

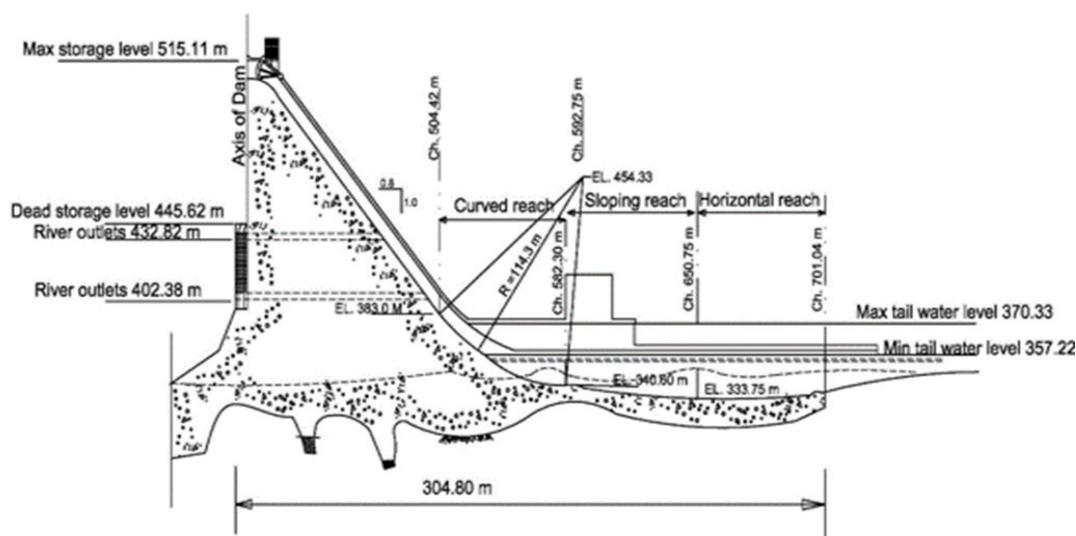


Figure 5-74: Bhakra dam, spillway's profile

- Cavitation

The process of cavitation in terminal structures can occur due to similar causes as in chutes. Pressure drop down to vapor pressure can be the result of high velocity flow over any irregularity or change at boundary, whose effect may be to converge the streamlines and to increase the local velocity. At low pressure, there may be bubbles (cavities) formation that travel in the water and rapidly collapse in a higher-pressure zone downstream, with damages to the concrete surface. Cavitation damages look different from that due to abrasion since it pits in an irregular pattern that cuts around aggregates of concrete, resulting in holes with rough edges. This erosive process can occur even with the passage of frequent floods smaller than IDF.

Typical locations of cavitation damages in a stilling basin are joints in the apron slab with some offset, entrance of chute and ancillary elements, floor and walls; in roller type energy dissipators, at any irregularity of the curved surface and at end lip, especially if it is slotted or dentate.

Once the process of cavitation is ongoing, time to serious damage up to failure of structural element will depend on duration and repetition of flow, and characteristics of concrete. As explained in chutes, potential

of cavitation is defined by cavitation index (see expression in Chapter 3). The classes of damage are related to the level of cavitation from incipient to severe damage, the threshold for concrete damage is named incipient damage, cavitation damage or beginning of damage. In energy dissipators, an estimated critical index is used according to the cause of cavitation; if calculated index at any location is lower than a critical index, the phenomenon occurs and damages appear.

The critical index is a typical value for each type of surface irregularity or obstacle; these indexes have been obtained in models for a determined configuration of boundary or shape of elements, so they are only a first reference to evaluate cavitation potential in an energy dissipator. Table 5-2 gives values for critical cavitation index for beginning of damages. It is important to mention that sometimes cavitation index in a stilling basin is calculated for entrance flow conditions and compared with a critical index for an element (i.e. baffle block) located some distance downstream, this practice is conservative since velocity and pressure in front of the block can be different as measured in models, so evaluation of cavitation and use of critical indexes should be done with caution.

Element or type of irregularity, on concrete surface	Critical Cavitation Index	Reference
Baffle pier pyramidal shape (recommended range)	1.4 to 2.3	Galperin et al (1977)
Baffle block as in stilling basin USBR Type III	0.33	Khatsuria (2000)
T shape baffle block	0.68	Kuttiammu (1951)
Abraded concrete surface with 20 mm of maximum roughness depth	0.60	Ball (1976)
Smooth changes in invert slope (1V:40H into or away from the flow)	0.20	Ball (1976) Arndt (1977) Falvey(1982)
Positive offset of 6 mm into the flow	1.6	-
Negative offset of 6 mm away from flow	1.0	-
End sill of stilling basins	1.05 to 1.75	-
Jet splitters in bucket	0.15 to 0.70	-
Spillway surfaces	0.20	Falvey (1982)

Table 5-2: Critical cavitation index for beginning of damages (ACI 210R98 and Khatsuria, 2000)

IS 12800-1989 (Indian Standard Criteria for Estimation of Aeration Demand for Spillway & Outlet Structures) may also be referred to in this connection. For stilling basin in outlet works, FEMA (2010) indicates the use of a critical cavitation index of 0.2 for basin without ancillary elements (blocks, sills) and 1.0 for basins with those elements; if cavitation index is lower than these figures, the basin is not appropriate for the flow conditions or the basin should be aerated. Cavitation damages that have occurred in some projects are shown in Figures 5-75 to 5- 78 for illustration.

- Hydrodynamic pressure fluctuations

The fluctuations of dynamic pressure constitute a hydraulic phenomenon linked to high velocity turbulent flow, being a special case, the macro turbulence of a hydraulic jump (as a large size vortex motion). This hydraulic load is present, typically, at the last sections of chute and in stilling basins. In bucket type energy dissipator, the dynamic pressure along the guided and deflected flow has lesser fluctuations. The hydraulic load comes from transformation of kinetic energy into pressure head in a significant percentage. The dynamic load changes direction on the slab surface; its action is both upwards and downwards. The results of this fluctuating load are: (1) Unequal negative pressure acting on the slab as an action of suction with potential vertical movement of the slab and (2) Sudden building up of uplift pressure also with potential of vertical displacement of the slab.

REFER TO APPENDIX A

FM12



Figure 5-75: Concrete surface of a baffle block with cavitation's damages (FEMA, 2010)

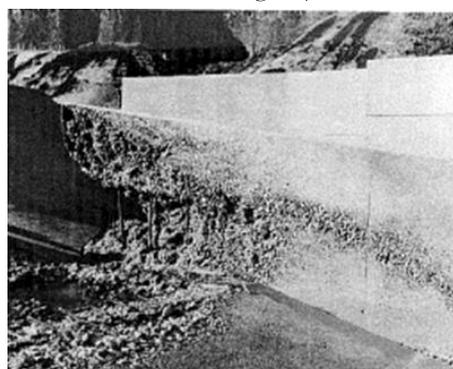


Figure 5-76: Cavitation damage in flip bucket and wall (ACI 210-R98)



Figure 5-77: Cavitation at bottom slab for  $V = 48$  m/s (3 m deep), Dworshak dam (USACE, 1980)



Figure 5-78: Cavitation damage downstream of chute blocks, Porto Combia dam, Brazil (ICOLD)

If the concrete surface has any opening or waterway through the slab as cracks, open joint, under drainage pipe discharge or any other, dynamic pressure fluctuations can propagate to the interface between concrete and foundation increasing uplift force above hydrostatic condition. Thus, there is a temporal and significant force acting on the slab that has to be balanced by the weight of the submerged concrete and by anchoring to the rock foundation. Depending on the amount of energy of incoming flow to the basin, the uplift “normally expected” (hydrostatic uplift controlled by tail water level) can be increased several times, with serious structural damages to the slab or even abrupt failure during a flood event (the slab is “turned over”).

As fluctuations of dynamic pressure are intense but random, the process is stochastic and ergodic; within the macro turbulence some slab panels could lose stability and get damaged or lifted up in the basin, while others being stable. In order to evaluate the

performance of the basin and its hydraulic safety, consideration of this destructive hydrodynamic load is fundamental. Even in case of new stilling basin, with adequate structural details, good finishing of concrete and sealed joints, it is important to highlight that underdrainage system discharging to the basin (drain pipes), could be the way to facilitate propagation of flow’s dynamic pressure to the foundation, in this case, the hazard increases if the hydraulic jump is drowned and moves towards the chute. In old structures the lack or loss of joint seal or aging of concrete increase potential of this process. Many reported damages and destruction of stilling basin’s structure have been due to this failure mode, most of them for discharges lower than design discharge.

Two others effects due to the dynamic forces can result: (1) Displacement of panel joints leading to an offset that induces cavitation, and (2) Vibration of structural elements which can produce cracks, displacement of joints and fatigue of reinforced concrete.

The fluctuations of pressure can be so severe that they could reach values below vapor pressure inducing instantaneous cavitation, which if repeated becomes a derived deleterious process. Also, structural vibration could be serious enough to produce fatigue of materials.

There are two methods of assessing hydrodynamic uplift, one based on measurement of fluctuating pressures with their spatial correlation and another based on direct measurement of fluctuating force (Khatsuria, 2005). Since research by Sanchez-Briebesca et al. (Mexico, 1979), there are different contributions for calculating uplift force and required thickness of concrete lining.

Toso et al. (1988) state that for practical purposes, the pressure fluctuations tend to approach a definite limit, of the order of

80–100% of the velocity head. The maximum pressure fluctuation are given by:

$$\Delta_p = C_p \cdot \frac{V_1^2}{2 \cdot g}$$

where,

$\Delta_p$  = maximum pressure fluctuation (deviation from the mean pressure) (m)

$C_p$  = coefficient of pressure fluctuations

$V_1$  = flow velocity of incoming flow (m/s)

$g$  = acceleration of gravity (m/s<sup>2</sup>)

This deviation pressure  $\Delta_p$  is assumed to act on the center of an area  $8 y_1$  by  $13 y_1$ . Moving out from the center of the area, the pressure would drop off to the mean pressure. Choose the smaller area between the actual slab area and the area  $8 y_1$  by  $13 y_1$ .

The values of  $C_p$  vary with the Froude number of incoming flow. For Froude number between 5 to 10,  $C_p$  varies from 0.9 to 1.2. (see Khatsuria, 2005).

The maximum expected uplift force acting on the center of a slab area is given by:

$$F' = \frac{1}{3} \cdot \Delta_p \cdot (\text{Area of slab}) \cdot \gamma_w$$

$$F' = \frac{1}{3} \cdot C_p \cdot \frac{v_1^2}{2g} \cdot (\text{Area of slab}) \cdot \gamma_w$$

where,

Area of slab = Width (B) times length (L) = smaller area between ( $8y_1$  by  $13y_1$ ) and actual slab size of basin (m<sup>2</sup>)

$\gamma_w$  = Specific weight of water (Kg/m<sup>3</sup> or kN/m<sup>3</sup>)

$F'$  = Uplift's force (Kg or KN)

In a simplified evaluation of a stilling basin slab with contraction joints pattern of B=6m, L=18m, a flow velocity of 25 m/s and assuming a mean value of  $C_p = 1$ , the pressure fluctuation is 31.85 m and the order of magnitude of the maximum uplift force at the center of panel,  $F' = 1,146,600$

kg (11.245kN) which is a large figure in comparison with the submerged weight of the concrete slab.

Other approaches (several based on direct measurements of force) gives lower values, so Toso equation is a first estimation of the acting force for evaluation purposes. For detailed analysis and for dynamic force acting on walls, specialized references must be consulted. IS-11527 “Structural Design of Energy Dissipaters” gives a methodology to estimate the uplift on account of hydrodynamic pressures below a stilling basin which is based on the work of Hajdin (1982). This is being used these days for Indian dams.

Figure 5-79 shows a historical case (1961) of damage of slabs in chute and stilling basin (breaking of concrete elements and foundation scour) due to combined hydraulic loads from cavitation plus hydrodynamic pressure fluctuations during a discharge of 20% of design (3,400 m<sup>3</sup>/s during 3 months), in Karnafuli's dam, height 41.2 m, Bangladesh. Figure 5-80 shows the energy dissipator of Bhamra Askhed dam (India) where concrete slabs were moved by hydraulic loads due to high velocity flow.



Figure 5-79: Damage due to cavitation + hydrodynamic pressure fluctuation, Karnafuli dam, Bangladesh.



Figure 5-80: Moved slabs by high velocity flow, Bhama Askhed dam, India

#### 5.5.4 Erosion downstream of terminal structures

The erosion process downstream of terminal structures could be an important hydraulic hazard to the spillway, to the dam (particularly for embankment dams) and for the river environment. The complexity of erosion is sensitive to the unitary specific flow,  $q$  ( $\text{m}^3/\text{s}/\text{m}$ ) and to the geological characteristics of riverbed material. Erosion can be present downstream of any type of existing terminal structure; the rate of erosion and the advance of the process vary with local river morphology and frequency of operation of spillway, and not only with the magnitude of discharge because serious damages have been reported with discharges much lower than design.

The common causes of erosion downstream of terminal structures are:

- In stilling basins and roller bucket type:
  - Remaining energy content in flow.
  - Malfunctioning of energy dissipator.
  - Unforeseen hydraulic behavior for low discharges.
  - Inadequate protection of downstream zone.
- In flip bucket, trajectory bucket.
  - Characteristics of the jet.

- Bed material in impingement site.
- Plunge pool depth.
- Location of erosion's with respect to river banks, dam, close-by installations, others.

Usually location and level of terminal structures of any type is associated with a stable foundation, mainly sound rock, with adequate physical condition and mechanical characteristics, but at downstream, where discharge zone, exit channel or plunge pool are located, materials can be soil or rock, with variable characteristics. On the other hand, erodibility (erosion potential) of a material, is a very sensitive property linked to the on-site condition of material such as layered ground, soil mixture and massive rock, so response to flow action could vary (Bollaert).

In existing stilling basins, roller bucket and impact basin, the transition zone or first stretch of the exit channel receives a flow with residual energy, turbulence and surge activity. A common layout downstream of the terminal structure consists of a channel with width equal to that of the spillway or wider with protected or lined section and invert adjusted to the levels of the terminal structure and exit channel downstream. Extended erosion could undermine the energy dissipator and the end walls.

According to the results of hydraulic analysis of the terminal structures, for the design or upgraded discharge, the evaluation of this transition should cover: (1) Type of protection and its stability, (2) If rip-rap is used (size, gradation and base filter) or (3) New protection or lining criteria for upgraded design conditions. Usually the protection material should be flexible and permeable, such as rock fragments. There are some empirical equations for rip-rap design for this specific location (FEMA, USACE, others); however, recommendations based on hydraulic models should be better due to complexity of flow regimen at this location.

For the exit channel the bottom and bank erosion depend on shear stress of flow, wave pattern, curves in channel, super elevation and other such factors. In cases, the channel material is soil (cohesive or non-cohesive), soft rock or a protection material, so erosion should be checked. For good quality rock, erosion is not expected in a low velocity flow (subcritical regime). The basic evaluation is to compare shear stress due to flow with the critical stress of the channel's material; if the flow stress is greater than that threshold value there will be erosion (Briaud, 2008) . The basic equation for shear stress in open channel is:

$$\tau_b = \gamma R S_e$$

where:

$\tau_b$  = shear stress on channel's bottom (kgf/m<sup>2</sup> or kN/m<sup>2</sup>)

$S_e$  = energy slope

$R$  = hydraulic radius for the flow (m)

$\gamma$  = unit weight of water (kgf/m<sup>3</sup> or kN/m<sup>3</sup>)

The values of critical shear stress for different materials is available in many technical references; Figure 5-81 presents relations to define a reference  $\tau_c$ .

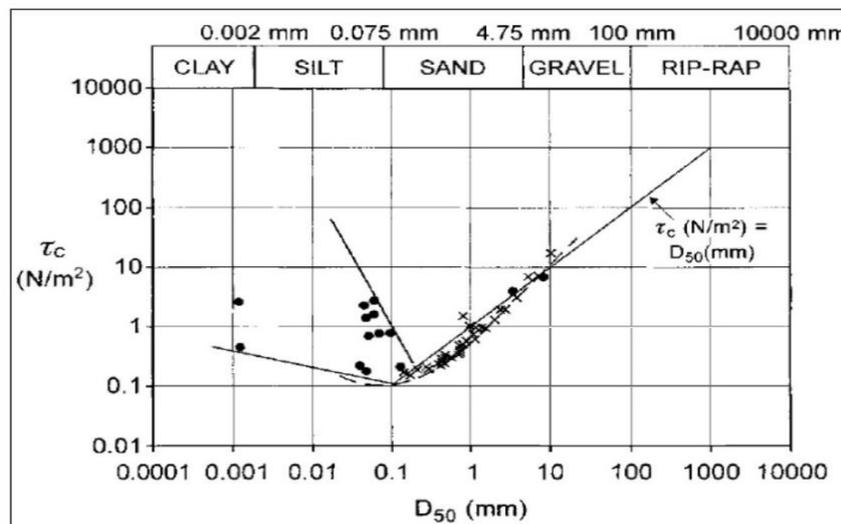


Figure 5-81: Critical shear stress vs. mean soil particle diameter of soil (Briaud, 2008)



Figure 5-82: Erosion damages in river's environment. Salal dam, India (Bhajantri et al.)

In general, erosion in rock at different physical conditions and with or without a sediment overburden, is related to:

- High velocity flow, as for example: unlined energy dissipating chutes or at the outlet of a spillway without

energy dissipator but a horizontal apron discharging directly to the bed rock.

- Impinging high energy jet on the bed rock as in a flip bucket or vertical water fall.

One of the credible and accepted Mode of Failure of a dam (especially for embankment dams) is due to erosion process and scour at its toe due to spillway discharge. Thus, evaluation of hydraulic safety is a matter of concern for the case of erosion downstream of flip bucket since the depth and extension of the affected area (erosion/scour hole) can endanger the dam, other appurtenant works and local environment. Figure 5-82 shows damages on river environment (bed, natural slopes and banks) due to impinging jet and water spray, during a large spillway discharge.

As mentioned previously in this chapter, rock erosion due to an impinging jet is a complex process that encompasses several parameters such as intensity of discharge, plunge pool and tail water level, rock condition and frequency of discharge. In order to estimate depth and extension of the erosion hole, three approaches are used:

- Empirical expressions developed based on laboratory tests and field data.
- Analytical-empirical expressions derived from laboratory and experiences on operative spillways.
- Mathematical models based on interpretation of scour mechanism due to jet pressure.

There are two aspects to mention about estimation of scour downstream of a flip bucket:

- The estimated scour by almost all expressions is the ultimate scour (steady condition of scour hole)
- The discharge used for estimation of scour should be lower than IDF - that is a representative flood event of the spillway operation, with lower return period.

REFER TO APPENDIX A

FM 8, FM 10, FM 12, FM 13, FM 14, FM 15 and FM 18

An approach to define that scour's formation flood is according to its probability of occurrence in the operative life of the reservoir. Some researchers suggest the use of a flood with an occurrence probability of 50% during reservoir life or a flood with return period from 50 to 100 years, but there is no international criteria.

Monitoring of the erosion process and its response to flood events, and changes in exposed rock condition is an important surveillance activity to be carried out in order to know about development of the erosion process and to predict its future behavior. This activity is especially required in large spillways.

Since researches of Schoklitsch (1932) and Veronesse (1937) and their formulae for scour, many expressions have been proposed based on different approaches, diverse applications and type of data (hydraulic and/or geotechnical). Empirical formula for ultimate scour has the form: (refer to Figure 5-29 for symbols/parameters):

$$Y = t + h = K * \frac{[H^y * q^x * h^w]}{g^v * d_m^z}$$

where:

Y = ultimate scour depth (m)

H = fall height (m)

q = specific or unit discharge (m<sup>3</sup>/s/m)

h = tailwater depth measured from initial river bed level (m)

t = scour depth below the initial bed level (m)

d<sub>m</sub> = characteristic sediment size or rock block diameter (m)

g = acceleration of gravity (m/s<sup>2</sup>)

K = constant

v, w, x, y, z = specific exponents for each formula, where:

Mason and Arumugam (1985), based on scour data, proposed the following values for calculating "Y" (Khatsuria, 2010):

- $K = 3.27$
- $v = 0.30$
- $w = 0.15$
- $x = 0.60$
- $y = 0.05$
- $z = 0.10$  and
- $ds = 0.25$  m (for rock fragments)

This formula is applicable for free jets issuing from flip buckets, pressure outlets and overflow works. It gives results with a standard deviation of the results of 30% for prototype test conditions. The applicability for the fall height  $H$  lies between 15.8 and 109 m for prototypes. It covers cohesive and non-cohesive granular models. Other jet parameters have been included in the determination of scour, for example, the jet impact angle; however, research has found that for angles of  $60^\circ$  to  $90^\circ$  (see Figure 5-29), which covers most of the angles encountered in practice for plunging jets, has negligible influence on the ultimate scour depth.

Due to the complexity of the scour process, selecting a appropriated formula, an ap-

proach or even a model, will depend on its application on each case. For a preliminary evaluation, a simple expression could be enough to quantify the potential or probable scour. Khatsuria (2010) and IS 7365:2010 propose the use of a Modified Damle empirical equation (1966), which is presented in Figure 5-83 by lines (named "A") for each probable depth of scour as:

- $A_1$ : Minimum expected scour,  $d_s = 0.36 (q h_o)^{0.5}$
- $A_2$ : Probable or reasonably expected scour under sustained spillway's operation,  $d_s = 0.54 (q h_o)^{0.5}$
- $A_3$ : Maximum or ultimate stabilized scour,  $d_s = 0.65 (q h_o)^{0.5}$
- $A_3$ -R: Modified Maximum scour,  $d_s = 0.90 (q h_o)^{0.5}$

$d_s$  = scour depth below tailwater level (m)

$q$  = unit discharge for discharge adopted for scour estimation ( $m^3/s/m$ )

$h_o$  = head from reservoir level to flip bucket lip level (m).

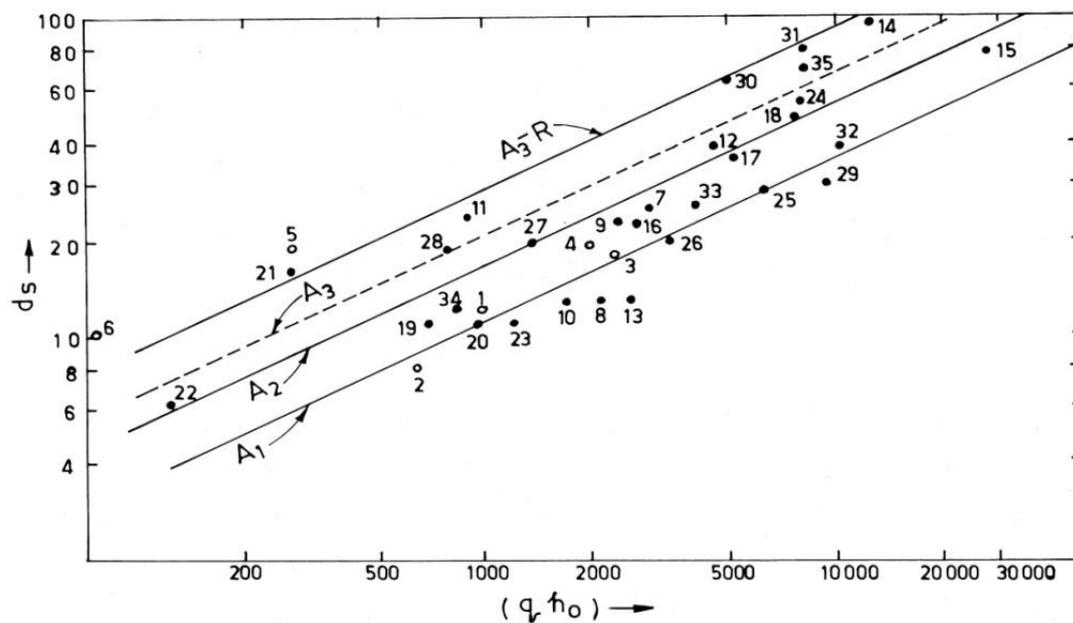


Figure 5-83: Scour downstream of flip bucket, Damle's equation (Khatsuria, 2010)

On Figure 5-83, there are 35 points from measurements at prototypes; several points are spillways from India. It has to be mentioned that some points on figure are not flip bucket but are free vertical water jet from an arch dam such as Kariba (number 14).

For the thirteen (13) spillways of India included in Khatsuria sample, details in respect of two dams is as under:

- $ds = 8$  m, number 2: Panchet Hill dam ( $Q = 17,853 \text{ m}^3/\text{s}$ ,  $h_0 = 45$  m,  $ds/h_0 = 0.18$ )
- $ds = 40$  m, number 12: Srisaillam dam ( $Q = 38,370 \text{ m}^3/\text{s}$ ,  $h_0 = 145$  m,  $ds/h_0 = 0.28$ )

The Table 5.3 presents the value “ $qh_0$ ” for 20 projects with flip buckets, some of them (8) are included in Figure 5-83 with its measured depth (in one or two dates). Figure 5-84 to 5-86 show erosion/scour in some dam

Sl. N° on Figure (*)	Project	$h_0$ (m)	$q$ ( $\text{m}^3/\text{s}/\text{m}$ )	$qh_0$
-	Banas	42.7	36.3	1,550
-	Bhatsai	47.9	85.6	4,100
-	Dimbhe	46.5	66.5	3,092
4, 7	Gandhi Sagar(**)	53.9	32.4	1,746
-	Gima	41.3	32.3	1,334
3	Hirakud	64.8	39.7	2,573
1	Maithon	85.7	38.1	3,265
5	Mandira	42.7	19.8	845
-	NagarjunaSagar	120	124.0	14,867
2	Panchet Hill	81.8	34.1	2,789
	Radhanagari	28.4	29.5	838
11	RanapratapSagar	61.1	31.4	1,919
-	Rihand	55.7	78.3	4,361
-	Salandi	29.3	34.1	999
-	SriramaSagar	59.4	27.7	1,645
-	Sukhi	44	24.8	1,091
-	Surya	44.2	48.2	2,130
6	Tilaiya	24.7	29	716
8, 9	Ukai (***)	97.6	47.9	4,675
-	Vaitarna	27.9	70.7	1,973
(*) Two numbers for a project - They are two dates of measurement of scour depth				
(**) Gandhi Sagar, measured from 1962 to 1973, 11 Years				
(***) Ukai, measured from 1973 to 1976, 3 years				

Table 5-3: Data of India's spillway with flip bucket to estimate expected scour

Example: the emblematic case of rock scour downstream of flip bucket: Wivenhoe dam, SE Queensland, Australia.

This is a relatively recent case (2011), which highlighted the uncertainty associated with the rock erosion process in a spillway with moderate head (30 m), provided with a de-

flector bucket and an excavated plunge pool in competent rock.

General data:

- Date of construction: Early 1980's
- Geology: Massive sandstone with reasonable widely-spaced horizontal joints.

- Hydraulic: Design discharge = 12,000 m<sup>3</sup>/s; erosion flood event (2011) = 7,500 m<sup>3</sup>/s and during four days the discharge was about 3,500 m<sup>3</sup>/s.
- Energy dissipator: Flip bucket with a pre-excavated plunge pool comprised benches stepping down in increments generally of 3m, up to a depth of 28 m below deflector's lip.

REFER TO APPENDIX B.1, B.3 and B.4

For developing and pre excavated plunge Pool

- Spillway and plunge pool: As shown in Figures 5-87 to 5-88.

The incident as reported by Lesleighter et al (2013): “Due to the high discharges occurring in tributaries downstream of the dam, the tailwater remained elevated for a number of days, and it was only when the tailwater subsided four days after passing the peak discharge that the top of an enormous rock mound that had developed in the spillway channel was observed. The pile of rocks were approximately 11m high, and nearly the full width of the channel. Boulders of up to 15m x 10m x 3m, weighing over 1,200 t were observed in the pile of eroded rocks in the spillway channel. The boulders appeared to have separated on the near-horizontal bedding planes. Bathymetric surveys of the plunge pool were undertaken in 2000 following the 1999 flood, and also in January 2011 after the flood, allowing for assessment of the progression of erosion over time. It can be seen that the majority of erosion which occurred during the January 2011 flood, as indicated in the longitudinal section, removed material from the downstream extent of the plunge pool to extend its length by more than 40 m and its base down to 2 m below design”. Figures 5-87to 5-89 complete the description of “almost unbelievable” quick and huge process of erosion in rock.

Later technical evaluation by Lesleighter, Stratford and Bollaert, highlights these findings: (1) Incident responded to 2000 years storm, (2) The hydraulic head 30 m is modest, the flow velocity at tailwater was 22 m/s and 10-12 m/s on rock surface, all these values are lower than other reported cases, (3) The fracturing and ejection of massive rock was dramatic, large rock blocks were moved almost 10 m vertically out of the pool, (4) Air entrained in the plunging jet has a definitive role in creating an unstable turbulence and hydrodynamic effects of massive rock, 5) Mechanisms acting on massive rock and procedures include as those in CSM (Bollaert,2012) showed success to represent the rock erosion process, (6) Erosion extended the plunge pool downstream but local undermining occurred in upstream slope.

Spillway concrete section had been stabilized by post-tensioned anchors in 2005, which ensure the dam against undermining, (8) Even though the CFD analysis provided a valuable appreciation of jet hydraulics, it has deficiencies thus the flow behavior is quite general and the hydrodynamic behavior of rock erosion is inadequate, (9) Physical model might be able to partially encompass those shortcomings of CFD, but the small scale of the model prevents from correctly reproducing the pressure fluctuations as well as the peak pressures. (10) “The Wivenhoe experience provides a valuable alert and shows that we cannot be sure about the ability of high-velocity flows to cause extensive scour even in what may be considered competent rock”. Others examples of scour in rock downstream of spillways in Australia, presented by Bollaert and Lesleighter (2014), are the following dams: Julius (1997), Burdekin (2009), Awoonga (2013) and Boondooma (2013).

REFER TO APPENDIX D

For Hydraulic Modelling



Figure 5-84: Erosion after a season of floods, Paradise and Borumba dams, Australia (Bollaert, et al, 2016 and 2014)



Figure 5-85: Scour of top layer of fractured rock and exposed anchors, after flood (Bollaert and Lesleighter, 2014)



Figure 5-86: Scour of river due to bottom sluice discharge, Balimela dam, head = 30 m,  $Q= 1,135 \text{ m}^3/\text{s}$

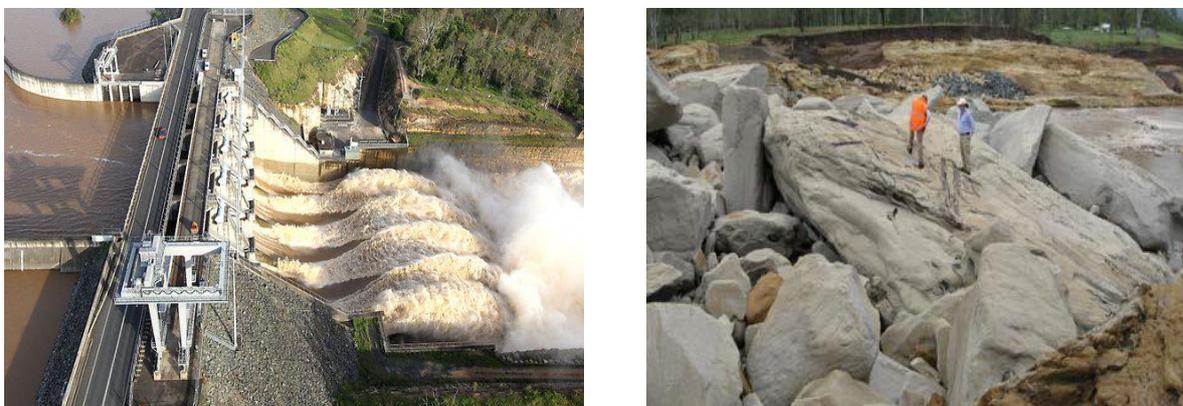


Figure 5-87: Operation during 2001 flood, rock mount-blocks up to  $15 \times 10 \times 3 \text{ m}$ , Wivenhoe dam, Australia (Lesleighter et al, 2013)  $Q= 1,135 \text{ m}^3/\text{s}$

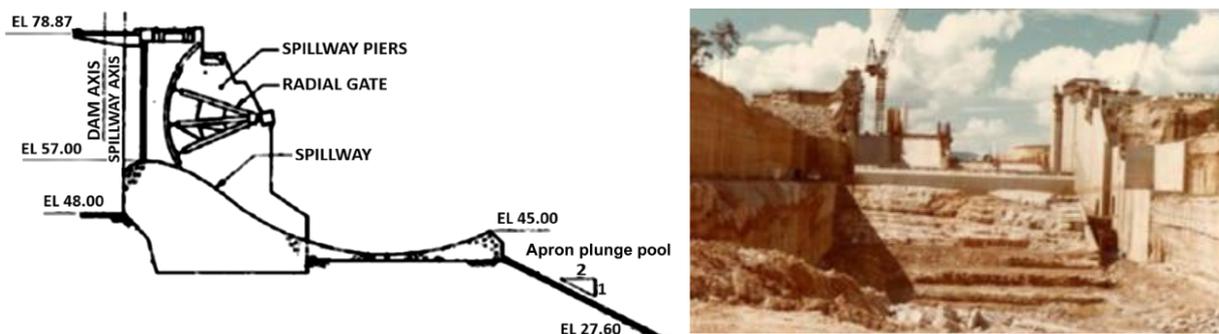


Figure 5-88: Flip bucket and excavated plunge pool, Wivenhoe dam, Australia (Leslighter et al, 2013)

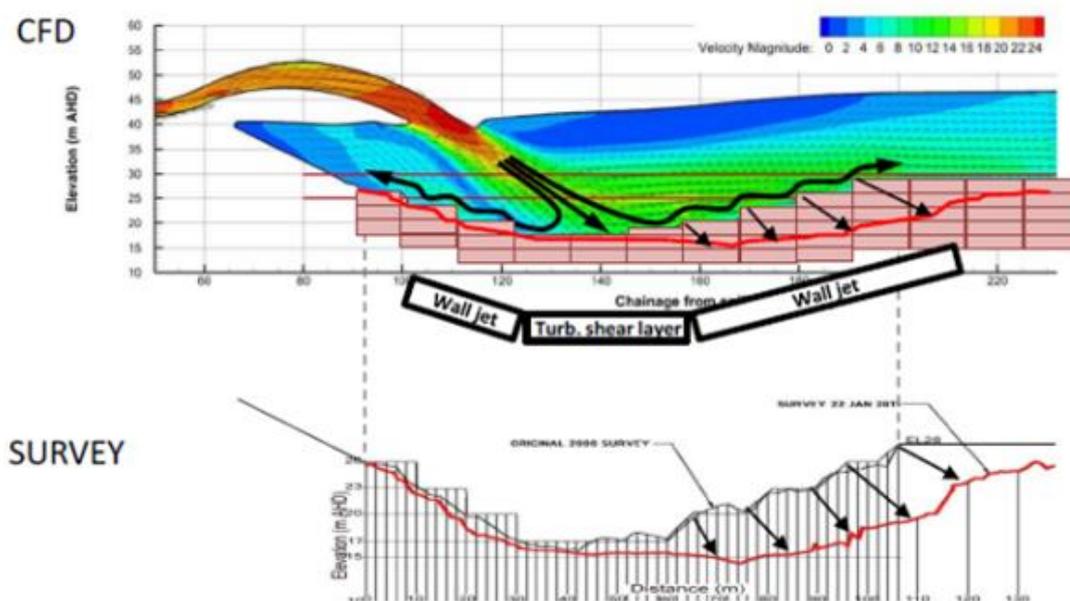


Figure 5-89: Plunge pool and rock erosion, CFD vs. bathymetrical survey, Wivenhoe dam, Australia (Leslighter et al, 2013)

REFER TO APPENDIX D

Numerical CFD Modeling in Hydraulic Structures

### 5.5.5 Tables with summary of assessment of hydraulic safety of terminal structures

While assessing the hydraulic safety of any component of a spillway, a key activity is to define credible modes of hydraulic malfunctioning and modes of failure of structural elements with loss of their function. As mentioned previously, these incidents may become a major operative hazard for the

dam-reservoir system with potential failure of the spillway and, eventually, of the dam.

The failure modes (FM) are based on the aspects presented throughout this chapter which cover hydraulic loads (change in discharge or hydraulic actions), surface conditions (damages to concrete surfaces) and flow regime (singularities). The response of the component or any of its elements defines whether the failure is limited to the component itself or it can extend to other components of the spillway, or to the dam. Appendix A covers the identification of FM for all components to be considered for a hydraulic safety assessment, including the terminal structures.

The following summary tables are guidelines for the evaluation of terminal structures and downstream zone (plunge pool and exit channel):

- Table 5-4: Aspects to be considered in the evaluation of terminal structures, plunge pool and exit channel of spillways.
- Table 5-5: Aspects to be considered in the evaluation of terminal structures,

plunge pool and exit channel of outlet works.

For the evaluation of these structures, the following CWC documents should be used together with this “Manual for Assessing Hydraulic Safety of Existing Dams”:

- A. Manual for Assessing Structural Safety of Existing Dams, CWC.
- B. Guidelines for Safety Inspection of Dams, CWC

Energy dissipator	Plunge pool and exit channel
(1) Tailwater rating Curve (TW vs. Q)	(1) Drag of loose material into dissipator
(2) Flow at the end of chute (WS profile)	(2) Back erosion and scour of structures
(3) Hydraulic jump characteristics	(3) Condition of cutoff at end of dissipator
(4) Apron (Invert) Level of Stilling basin	(4) Potential regressive erosion (head cutting)
(5) Length of Stilling basin	(5) Depth of plunge pool
(6) Hydraulic jump’s stability	(6) Extension of plunge pool
(7) Freeboard of walls in a stilling basin	(7) Bank erosion in pool or river banks
(8) Geometry of ancillary features for new Q	(8) Bars formation / obstruction downstream
(9) Erosion at basin’s outlet	(9) Water surface (WS) profile in exit channel
(10) Hydrostatic uplift due to TW for new Q	(10) Capacity and erosion at exit channel
(11) Hydrodynamic pressure fluctuation	(11) Effect of pool in dam/near structures
(12) Geometry of bucket for new Q	(12) Frequency of site inspection
(13) Level of bucket lip vs. TW	(13) Environmental impact at river
(14) Performance of roller bucket, Q and TW	(14) Effect of sediment discharge
(15) Trajectory of jet from flip bucket	(15) Geological features: rock at plunge pool
(16) Stability of bucket: for new Q	
(17) Abrasion	
(18) Cavitation	
(19) Asymmetrical distribution of incoming flow	
(20) Surface and details of concrete elements	

Table 5-4: Aspects to evaluate of terminal structures, plunge pool and exit channel of spillways

Energy dissipator	Plunge pool and exit channel
(1) Tailwater rating Curve (TW vs. Q)	(1) Drag of loose material into dissipator
(2) Flow’s conditions at the inlet	(2) Bank erosion and scour of structures
(3) Hydraulic jump characteristics	(3) Condition of cutoff at end of basin
(4) Apron (Invert) Level of stilling basin	(4) Depth of plunge pool
(5) Length of basin	(5) Extension of plunge pool
(6) Hydraulic jump’s stability	(6) Bank erosion in pool or river banks
(7) Freeboard of walls of stilling basin	(7) Obstruction downstream
(8) Geometry of basin and ancillary features	(8) Water surface profile in exit channel
(9) Erosion at basin’s outlet	(9) Capacity and erosion in exit channel

Energy dissipator	Plunge pool and exit channel
(10) Hydrostatic uplift due to TW	(10) Effect of pool in dam/near structures
(11) Hydrodynamic pressure fluctuation	(11) Performance of outlet valve
(12) Level of end lip of bucket vs. TW	(12) Frequency of inspection of valve
(13) Trajectory of jet from flip bucket	(13) Frequency of inspection of discharge area
(14) Stability of bucket	(14) Environmental impact at river
(15) Abrasion	(15) Effect of sediment discharge (sluices)
(16) Cavitation	
(17) Surface and details of concrete elements	

Table 5-5: Aspects to evaluate of terminal structures and exit channel of outlet works

## 5.6 Rehabilitation Measures for Terminal Structures

This section presents measures for upgrading the energy dissipators and the exit channels of existing spillways, divided in two groups:

- Measures to adapt the terminal structures to an increase in discharge
- Measures to improve performance and to avoid damages by hydraulic loads in terminal structures

Another CWC publication “Guidelines for Rehabilitation of Large Dams” covers measures for repairing the structural components of the spillway. Inspection and maintenance activities are not in the scope of this Manual, but the following publications can be referred for these topics: “Guidelines for Safety Inspection of Dams” and “Guidelines for preparing Operation and Maintenance Manual for Dams”. Aspects related with structural behavior and repairing measures of appurtenances works are covered in: “Manual for Assessment of Structural Safety of Dams”.

### 5.6.1 Measures to adapt the terminal structures to an increase in discharge

The spillway capacity, as described previously, can be increased by various structural/non-structural measures.

In the following two cases, the terminal structure and exit channel are required to function with more discharge:

- Case 1: Accepting a higher MWL without any change in spillway in cases where the reduced freeboard of the dam above that revised MWL is adequate.
- Case 2: Upgrading of an existing spillway by conversion of broad crest into ogee crest or conversion of an un-gated spillway into a non-conventional spillway like Labyrinth, PKW, etc.

Once the terminal structures have been evaluated for the new discharge, rehabilitation measures could vary depending on the increase in discharge and the type of energy dissipator. There is no specific criteria about how much more discharge can be handled by the same terminal structure; hydraulic evaluation, commonly based on physical and numerical models, is the most convenient approach to make a decision. Another aspect to consider is whether some damage can be accepted in the terminal structure and exit channel during occurrence of a high flood. This last decision has to do with management of risk, and will depend on the type of spillway (main, auxiliary or emergency), cost, level of damages and consequences of spillway on reservoir operation and environment.

In qualitative terms, the rehabilitation measures could be as follows:

- For low/marginal increase in discharge: This case envisages use of some limited measures in the terminal structures, accepting unsatisfactory hydraulic performance and some damage, but not structural instability.
- For significant increase of discharge (with same spillway or upgrading the same).

The following example allows to visualize the effects of increase in discharge in a spillway due to hydrological upgrading of IDF, and its effect in hydraulic condition for the terminal structure.

Assuming that the control structure is a free (ungated) ogee crest with an effective length of 100 meters, a design head of 4 m and a mean coefficient of discharge “C” of 2; with same width of chute and energy dissipator, 100 m.

The original capacity is given by:

$Q = CLH^{3/2} = 1,600 \text{ m}^3/\text{s}$  with a unit discharge of  $16 \text{ m}^3/\text{s}/\text{m}$ . The following two options are discussed:

Options:

- (1) To accept an additional 1 m increase in reservoir water level (MWL) (subject to adequacy of reduced freeboard) to spill

part of new IDF and to incorporate an emergency spillway for excess discharge.

- (2) To change the ungated weir to a PKW with capacity for new IDF.

Table 5-6 presents results of the two approaches to increase spill capacity; it may be seen that the resulting unit discharge entering in terminal structure is quite different in both cases.

It is important to consider that the terminal structures and exit channel could be required to function efficiently up to dominant discharges (a % of the design capacity of the spillway), to be decided by factors such as maximum observed flood). As a matter of fact, in India, IS-11223 (R2004) includes a “Flood, Inflow Design (IDF), for efficient operation of energy dissipation works” which is “a flood which may be lower than the inflow design flood for the safety of the dam. When this flood is used with standard specifications or other factors affecting the performance, the energy dissipation arrangements are expected to work most efficiently. No damage/breaches in the breaching section, fuse plug, etc, are contemplated during this flood”.

CASE	Increase in discharge (in m <sup>3</sup> /s)	Increase in discharge (in %)	Upgraded Q (m <sup>3</sup> /s)	Updated Unit discharge q (m <sup>3</sup> /s/m)	Increase in Unit Discharge q (energy content) (in %)
Case 1: +1 m MWL	636 m <sup>3</sup> /s (*)	39.75%	2,236	22.36	39.75%
Case 2: using a PKW	3200m <sup>3</sup> /s (**)	200 %	4,800	48.0	200 %

(\*) Calculated by the formula,  $Q = CLH^{3/2}$  taking  $C = 2$ ,  $L = 100 \text{ m}$  and  $H = 5 \text{ m}$ .  
 (The original discharge with  $C = 2$ ,  $L = 100 \text{ m}$ ,  $H = 4 \text{ m}$  was 1,600 cumec).

(\*\*) Mean discharge with a PKW of same physical length is three times of a conventional free ogee crest weir.

Table 5-6: Example of increased discharge in a spillway and new hydraulic condition for terminal structure

### **5.6.2 Measures to improve performance and to avoid damages in terminal structures**

Measures to improve performance of existing (operative and aged) terminal structures are not to be restricted to guarantee the hydraulic behavior only but also to ensuring their structural soundness. In these cases, the past emergencies, previous evaluation or risk studies would have already alerted about the different levels of hazardous or unexpected hydraulic loads. The expected response may vary from incipient damage as indicator of inadequate functioning to loss of function (of an element or the whole structure).

These measures are applicable both to terminal structures with required spillway capacity and also for those which are to be adapted to the new increased spillway discharge.

In this manual, various hydraulic actions are defined and adverse structural features and such details are highlighted; however, the structural actions to remediate and solve the problems may be carried out as per procedures, techniques and materials presented in “Manual for Assessment of Structural Safety of Dams” and “Guidelines for Rehabilitation of Large Dams” and various other references and standards available.

### **5.6.3 Tables of rehabilitation measures of terminal structures and exit channel**

The rehabilitation measures proposed for terminal structures are focused in increasing their capacity or improving hydraulic performance, in order to reach a level of hydraulic safety that conforms to an “appropriate hydraulic behavior”; all within the general safety and risk management for dam-reservoir system.

As mentioned previously, the concept of “*Flood, Inflow Design (IDF), for efficient operation*

*of energy dissipation works*” (as for IS-11223-2004) could be considered during evaluation and selection of rehabilitation measures.

As mentioned, in Chapter 3 and in this chapter, hydraulic safety encompasses a group of conditions or loads that affect structural elements of the appurtenant works leading to progressive damage, abrupt break of an element and failure of a component of the terminal structures. Commonly, the worst consequence for the dam-reservoir system is a temporary loss of an operative spillway, the safety of the dam itself is not directly affected; however, there are reported cases of total loss of the spillway with uncontrolled discharge to downstream zone, loss of reservoir purposes for a long time and break of embankments dams due to erosion at abutment or river bed (see Appendix A Case of Studies). For outlet works, hydraulic safety is focused in guarantying normal operation according to the purpose of the reservoir during an emergency as complementary spillway to cope with high flood events or when the drawdown of the reservoir level is required.

The rehabilitation measures require to be studied and designed according to the type of hydraulic problem, existing conditions of structural element, engineering criteria, materials, construction methods and cost. For new designs, they should conform to Indian (BIS) and International Standards, and use local and recent materials and appropriate construction technology. The rehabilitation measures, require a detailed knowledge of the problem based on sound inspection and investigation of each case.

Tables 5-7, 5-8 and 5-9 present the rehabilitation measures for terminal structures and exit channel for spillways grouped as:

- Table 5-7 - For increasing terminal structure capacity for the following cases:
  - CASE 1: Limited increase in discharge (Table 5-7(A)).

- CASE 2: Significant increase in discharge (Table 5-7(B)).
- Table 5-8 - For improving hydraulic performance and for avoiding major damages due to abrasion, cavitation, uplift, hydrodynamic pressures or other causes, in:
  - A: Stilling basins (Table 5-8(A)).
  - B: Flip bucket (Table 5-8(B)).
- Table 5-9 - For improving hydraulic performance and for avoiding major damages, mainly due to erosion, in:
  - Exit channel, plunge pool and river environment

It is important to mention that for outlet works, the increase in discharge applies only to those terminal structures related with intermediate or bottom outlets whose main function is to complement capacity of the spillway. These outlet works are sometimes provided as sluices through a masonry/ concrete dam which discharge into the same energy dissipator as that of the surface spillway e.g. Hirakud dam, so its influence in changes in that structure could be minor; however, an important aspect to consider with flow through sluices and bottom outlet located near the river bed is the sediment load (sand, gravel, boulders) and its effect in concrete damages at the energy dissipator.

Other functions of the outlet works do not imply any significant changes in discharge and in hydraulic conditions with small increases in water level of the reservoir.

## 5.7 Lessons

From the contents of this chapter, several aspects could be considered as lessons from terminal structures performance:

Energy dissipation:

- IS-11223 (R2004) includes a “*Flood, Inflow Design (IDF), for efficient operation of energy dissipation works*”. Evaluation and rehabilitation of terminal structures and exit

channel could be, in some cases, associated to this flood event lower than IDF (as a % of IDF). For that discharge, the energy dissipation arrangements are expected to work most efficiently.

- Energy dissipation in large dams is normally accompanied with large turbulence, high gradients and energy, high flow concentrations, low pressures and high-pressure pulsations, two phase flow of different types - mixing takes place either in the air or in the water. Flow is largely destructive.
- Frequent performance of submerged energy dissipators, include introduction of material (sand, gravel, rock fragments) into the basin, thus creating large damage due to abrasion.
- Large hydro dynamic pressures/forces may get generated in existing dams, which were not contemplated in original designs.
- There are many cases where IDF review shows the design flood to have doubled or even more with respect to the original value. The energy dissipators are required to be checked for larger than original design flows. It is quite possible that in the case of flip buckets the trajectories may be shorter than original and in stilling basins the hydraulic jump may get swept out of the basin, more frequently. Aerating devices may prove insufficient. Tailwater levels may be higher than expected. All these effects may constitute hazards for the performance of the energy dissipators which needs to be closely evaluated.
- Obviously, submergence of energy dissipators create a problem for maintenance and inspection tasks, inspections are difficult and due to extremely large volume of water to be pumped out. Other options include use of professional divers or ROV for underwater survey. However, these options are expensive.
- Physical hydraulic and CFD mathematical models are fundamental tools to visualize and to analyze flow behavior, energy dis-

sipation, prediction of erosion and related hydraulic hazards for terminal structures. For rock scour predictions and effect of air entrainment, there are still limitations with the use of models.

- Slotted Roller buckets are these days not favored as their teeth are prone to damages.

Stilling basins:

- The hydraulic functioning of stilling basins has to be checked for ample range of discharges, not only the design discharge. Hazardous conditions may arise with low discharges; as a matter of fact, many damages have been reported with frequent discharges much lower than design.
- Stilling basins are sensitive to variations in tailwater. Definition of tail water and location of hydraulic jump is an important task in evaluation of this type of dissipator, also there can be potential changes in tail water rating curve due to erosion or obstruction of exit channel.
- Stilling basin for very large heads (higher than 120-150 m) are seldom used; however, Tehri Dam and Bakhra Dam in India, are examples of stilling basins in operation with more than 250 m of head. Some other previous prototypes have not performed so well and have been subject to costly repair works (Tarbela, Malpas, others) or even large upgrades (Sayano Sushenskaya).
- In large stilling basins, poor performance is due to several factors, abrasion due to introduction of loose material by return flows and/or from sliding material from basin banks, cavitation damage due to existence of localized low pressures at boundaries or at ancillary elements, uplift due to pulsating high hydrodynamic loads enhanced by poor construction practices.

Flip buckets:

- Flip buckets are preferred as energy dissipators for cases involving large heads and large unitary flows (intensity of dis-

charge), provided good geology is available in the river bed.

- Prediction of rock scour due to highly aerated water jet, is still a matter of investigations. Even in competent rock, the hydrodynamics effects involved in an impinging jet can cause dramatic scour. As Bollaert states: "Sometimes the rock is strong and apparently of good quality. Nonetheless, in virtually all situations major scour has occurred. This is a world-wide experience".
- In the plunge pool several kinds of flow conditions can be observed, wave uprush, lateral and back currents, large scale circulations, vortex, jet impact, high content of air-water in contact with bank boundaries creating humidity and potential conditions for river banks to slide, high capacity of flow to transport rock elements inside the pool.
- Medium head and even large specific flow are suitable for flip buckets, but a pre-excavated plunge pool is recommended to improve energy dissipation.
- Submerged operations (i.e. bucket lip submerged by tail water) of flip bucket can be random with frequency. There can be instances, when the bucket may perform poorly during small discharges, resulting in significant damages involving costly reparation, even though it may have safely passed the floods of several years return period earlier. Free flip buckets will permit ease access and inspection.
- Submerged flip bucket combined with sediment-laden flow is a highly detrimental mode of operation. This situation is presented in several orifice/bottom spillway in run-off river projects located in mountain regions.
- Horizontal or mild sloped lip of a submerged flip bucket, poses great difficulty requiring artificial flow aeration to protect the concrete surface against cavitation damage, since return flow may make the air cavity to collapse thus creating very difficult conditions for the air to access

- the cavity, resulting in poor aeration of the flow and eventual cavitation damage.
- Asymmetrical flow in the plunge pool and exit channel can create many problems downstream such as large scoured areas, damage to close by infrastructure, also formation of bars in the exit channel which may increase submergence.
- For high flow velocities (higher than 25-30m/s),the exit boundaries such as flip bucket lip, are normally subject to minor cavitation damages. At these areas flow instability is frequent thus generating a localized spot with pressure pulsating including negative pressures.

Rehabilitation measures for increasing terminal structure capacity in spillways		
CASE 1: Limited increase		
Hydraulic Action	Hydraulic requirement /condition of structural element	Rehabilitation Measure
Stilling basins Increase in energy content at entry with changes in hydraulic jump's characteristics	Higher conjugate depth and TW level requirement. Need of higher walls.	Accept partial encroachment in free-board of basin's walls. Increase of wall height with complementary concrete elements. Accept some wall's overtopping and use of fill surface protection. Check for hydrostatic uplift due to greater tail water and use structural measures to guarantee stability.
	Larger length of hydraulic jump. Need to increase the basin length	Accept movement of the hydraulic jump, possible sweep-out from basin. Incorporate a weir downstream with additional energy dissipation to increase tail water level. Improve protection downstream of basin. Modify ancillary elements if needed: chute blocks, baffle and end sill.
	New hydraulic conditions at stilling basin. Need to check potential hydraulic loads and concrete conditions.	Hydraulic evaluation of stilling basin. Hydrodynamic pressure fluctuations. Inclusion of performance or security measures as presented latter.
Flip buckets Increase in energy content of entrance flow and changes in jet characteristics of the jet	Changes in jet trajectory. Displacement of impingement site. Larger depths and gravity effects, jets may fall near the concrete structures.	Adapt plunge pool area. Protection or reinforcement of impingement site. Protection of river banks and close-by installations. Test flip bucket performance for larger flows in physical models
	Higher tailwater depth, possible submergence of flip bucket	After hydraulic evaluation, accept some level submergence of deflector. Local protection downstream of flip bucket: concrete slab, heavy rip-rap, etc. Test operation rules of the spillway to operate the bucket free of submergence
	New hydraulic conditions at bucket. Need to check potential hydraulic loads and concrete conditions.	Evaluation of geometry (radius, throw angle, levels) and inclusion of performance or security measures as presented latter.

Table 5-7 (A): CASE 1:Rehabilitation measures for increasing terminal structure capacity in spillways (Limited Increase)

Rehabilitation measures for increasing terminal structure capacity in spillways		
CASE 2: Significant increase		
Hydraulic action	Hydraulic requirement /condition of structural element	Rehabilitation measure
Stilling basins Increase in energy content of entrance flow and changes in hydraulic jump's characteristics	High conjugate depth and TW level requirement. Need of higher walls.	Increase of wall height according to hydraulic loads. Check tail water level for new discharge. Incorporate a weir downstream with additional energy dissipation to increase tail water level.  Check for hydrostatic uplift due to greater tail water and use structural measures to guarantee stability.
	Larger hydraulic jump. Need to increase the basin length	Lengthen the basin. Protection downstream of basin. Modify functioning of basin with ancillary elements: chute blocks, baffle and end sill. Adapt walls, transition discharge area and exit channel to new basin. Check for a cutoff element.  New stilling basin.
	New hydraulic conditions at basin. Need to check potential hydraulic loads and concrete conditions.	Evaluation and inclusion of performance or security measures as presented latter, especially, joint between structures, anchoring, under drainage system.
Flip buckets Increase in energy content of entrance flow and changes in jet's characteristics	Changes in the trajectory of water jet. Displacement of impingement site.	Adapt plunge pool area. Incorporate an excavated plunge pool. Protection or reinforcement of impingement site. Protection of river banks and close-by installations.
	Higher tailwater depth, possible flip bucket submergence	Check level of jet submergence and bucket performance. Increase wall height according to hydraulic loads. Protection of downstream of flip bucket: concrete slab, rip-rap, etc.
	New hydraulic conditions at bucket. Need to check potential hydraulic loads and concrete conditions.	Evaluation of geometry (radius, throw angle, levels) and inclusion of performance or security measures as presented latter. Inclusion of stability measures for structure as: anchoring, mass rock strengthening, cutoff, other.

Table 5-7 (B): CASE 2: Rehabilitation measures for increasing terminal structure capacity in spillways (Significant increase)

Rehabilitation measures for improving performance and avoiding major damages		
STILLING BASINS		
Hydraulic action	Hydraulic requirement /condition of structural element	Rehabilitation measure
<p>High velocity, rough flow in a stilling basin and its ancillary elements, with actual or upgraded discharge.</p> <p>Entrance of solids as: sand, gravel, boulders, concrete fragments, trees, other debris, from downstream channel or other origin.</p> <p>Abrasion</p>	<p>Damage to concrete, surface of slab/walls and steel bars reinforcement. Large abrasion damage. Potential slab break. In extreme cases partial or total loss of slab.</p> <p>Increasing possibility of occurrence of cavitation.</p>	<p>Draining and emptying of stilling basin for inspection and to access and to work in dry environment.</p> <p>Execute physical model tests</p> <p>Repair concrete surfaces. Materials for repairs of surfaces: steel, high performance concrete, etc.</p> <p>Investigate in Model tests, level of downstream surface (exit channel) required to guarantee that end sill is high enough to avoid material to get in the basin, due to any return flows (as required, depending on actual site conditions). Rehabilitate basin as appropriate.</p> <p>For technical procedures and materials, see Manuals “Assessing Structural Safety of Dams”, “Rehabilitation of Large Dams” and other technical references.</p>
<p>High velocity , rough flow in a stilling basin and its ancillary elements, with actual or upgraded discharge.</p> <p>Irregularities in concrete surface.</p> <p>Separation zones in ancillary elements</p> <p>Cavitation</p>	<p>Damage to concrete, surface of slab/walls and steel bars reinforcement. Potential slab break. In extreme cases movement of concrete fragments and collapse of slab.</p> <p>Potential erosion of foundation material.</p>	<p>Draining and emptying of stilling basin for inspection and to access and work in dry environment.</p> <p>Execute physical model tests. Repair concrete surfaces: roughness, offsets, displacements, other. Repair structural details such as: joints, sealing of joints (water-stops or specific filling products).For technical procedures and materials, see Manuals “Assessing Structural Safety of Dams” and “Rehabilitation of Large Dams” and other technical references.</p>
<p>High Turbulent, high velocity flow in a stilling basin and its ancillary elements, with actual or upgraded discharge.</p> <p>Hydrodynamic pressures fluctuations.</p> <p>Openings in slab: cracks, joints, drains. Suction forces on slab Increase of uplift force due to propagation of hydrodynamic pressure fluctuations to foundation.</p>	<p>Loss of stability of slab and increased load on walls.</p> <p>Cracking or breaking of slab panels. Sudden vertical movement of slab.</p> <p>Collapse of parts or whole slab, especially at the upstream part of basin or end of chute.</p> <p>Potential erosion of foundation material.</p>	<p>Draining and emptying of stilling basin for inspection and to access and work in dry environment.</p> <p>Detailed structural evaluation of concrete elements under hydrodynamic pressure fluctuation. Concrete surface restoration: cracks, open joints, other damages. Restoration of stilling basin slab with increased concrete thickness. Anchoring of stilling basin slab as needed. For technical procedures and materials, see Manual “Assessing Structural Safety of Dams” and “Rehabilitation of Large Dams” and other technical references.</p>

Table 5-8 (A): A Rehabilitation measures for improving performance and avoiding major damages in stilling basins

Rehabilitation measures for improving performance and avoiding major damages		
Flip buckets		
Hydraulic action	Hydraulic requirement /condition of structural element	Rehabilitation measure
High velocity flow in a flip bucket, with actual or up-graded discharge.	Damage to concrete surface of bucket/walls and reinforcement. Increasing possibility of occurrence of cavitation.	Concrete surface restoration and protection with specific material and technology. For technical procedures and materials, see Manuals “Assessing Structural Safety of Existing Dams” and “Rehabilitation of Large Dams” and other technical references.
High velocity flow in a flip bucket, with actual or up-graded discharge. Irregularities in concrete surface. Separation zones in ancillary elements at bucket lip Cavitation.	Damage to concrete surface of bucket/walls and reinforcement. Damage at end lip and dentate deflector. Effect in shape of the jet or mass water distribution	Repair or improve concrete surfaces Protect end lip with other materials— <del>steel</del> Check inclusion of forced aeration. For technical procedures and materials, see Manuals “Assessing Structural Safety of Existing Dams” and “Rehabilitation of Large Dams” and other technical references.
Turbulent high velocity flow in a flip bucket. Hydrodynamic pressures.	Stability of concrete structure of bucket/walls under hydrodynamic pressure.	Check structural behavior of walls Improving stability of bucket by anchoring to the rock. For technical procedures and materials, see Manuals “Assessing Structural Safety of Existing Dams” and “Rehabilitation of Large Dams” and other technical references.

Table 5-8(B): Rehabilitation measures for improving performance and avoiding major damages in flip buckets

Rehabilitation measures for improving performance and avoiding major damages		
Exit channel, plunge pool and downstream river environment		
Hydraulic action	Hydraulic requirement /condition of structural element	Rehabilitation measure
Irregular , turbulent flow at the transition between stilling basin and exit channel for actual or up-graded discharge.	Extended erosion. Back erosion and undermining of energy dissipator. Drag of material into the basin.	Execute physical models Geometry adjustment: shape according to eddies formation, adjust bed level at contact with end sill of basin, width and length. Bottom protection: rip-rap, concrete, others. Banks protection: rip-rap, gabions, others.
Actual or upgraded discharge vs. exit channel capacity.	Modification of tailwater elevation. Overflow of exit channel. Erosion at exit channel bottom	Occurrence of a downstream flow control section: evaluation and development of tail water rating curve. Removal of obstructions or potential loss of area of exit channel, vegetation etc. To

Rehabilitation measures for improving performance and avoiding major damages Exit channel, plunge pool and downstream river environment		
Hydraulic action	Hydraulic requirement /condition of structural element	Rehabilitation measure
	and/or banks.	carry out slope stability as required, protection of local installations, etc. Increase of channel capacity by enlarging its section , use of lining, erosion protection: rip-rap, gabions, others.
Overflow of lip to foundation for low discharges	Local erosion and scour. Potential scour of terminal structure/loss of stability.	Local protection with concrete slab. Reinforce or strengthening of rock mass: anchors, filling of cracks, dental concrete, etc.
Impingement site with or without a plunge pool.	Local erosion and scour. Back erosion towards the terminal structure and potential undermining. River banks erosion and loss of slope stability Deleterious water spray in the zone of discharge.	Protection of site from jet action: reinforced concrete (bed or slopes), closed to the bucket. Aeration by splitting the jet with deflectors at bucket lip or with aerators at the chute. Air content reduces scouring capacity of the jet. Excavate a plunge pool. Create a plunge pool by adding a tail pond dam downstream. Verify in physical model the efficacy of pool's level, Combination: excavated pool and tail dam Add a cutoff structure at the front face of flip bucket as anchored slab over rock. Reinforce or strengthening of rock: anchors, filling of cracks, etc. Geotechnical measures: river banks, slopes.
Transport and sedimentation of eroded material from impingement site or plunge pool.	Local changes in river morphology. Creation of bars of sediment, obstruction of the watercourse and effect in other installations.	River training and protection to avoid erosion of banks/bed by meandering channel. Channel with enough sediment transport capacity and excavated pool.

Table 5-9: Rehabilitation measures for improving performance and avoiding major damages in exit channel, plunge pool and river environment



## References

### Chapter 1

- ANCOLD. 2015. “Flood Evaluation and Dam Safety”. Australian National Committee on Large Dams. Hobart, Australia.
- BIS. 1985. “Guidelines for Fixing Spillway Capacity. Bureau of Indian Standards (BIS)”, – IS 11223-1985, Govt. of India, 10 pages
- Carrillo, J.M. 2009. “Evaluación de la Seguridad Hidrológico-Hidráulica de la Presa de Compuerta (T.M. de Velilla del río Carrión, Provincia de Palencia) perteneciente a la Cuenca Hidrográfica del Duero”. Universidad Politécnica de Valencia, España.
- CDA. 2007. “Dam Safety Guidelines”. Canadian Dam Association, ACB.
- CDGB. 2002. “Large Brazilian Spillways”, Foz de Iguazú, Brasil.
- CWC. 2017. “National Register of Dams”, Central Water Commission, Ministry of Water Resources, River Development and Ganga Rejuvenation, Government of India.
- CWC. 2019. “Guidelines for Assessing and managing Risks associated with Dams”, DRIP, Central Water Commission, Ministry of Water Resources, River Development and Ganga Rejuvenation, Government of India.
- CWC. 2019. “Guidelines for Safety Inspection of Dams”, DRIP, Central Water Commission, Ministry of Water Resources, River Development and Ganga Rejuvenation, Government of India.
- CWC. 2019. “Manual for Rehabilitation of Existing Dams”, DRIP, Central Water Commission, Ministry of Water Resources, River Development and Ganga Rejuvenation, Government of India.
- CWC. 2019. “Guidelines for preparing Operation & Maintenance Manuals for Dams “, DRIP, Central Water Commission, Ministry of Water Resources, River Development and Ganga Rejuvenation, Government of India.
- CWC. 2019. “Guidelines for Development and Implementation Emergency Action Plans for Dams”, DRIP, Central Water Commission, Ministry of Water Resources, River Development and Ganga Rejuvenation, Government of India.
- CWC. 2018. “Handbook for Assessing and managing Reservoir Sedimentation”, DRIP, Central Water Commission, Ministry of Water Resources, River Development and Ganga Rejuvenation, Government of India.
- CWC. 2018. “Manual for Assessing the Structural Safety of Dams, DRIP, Ministry of Water Resources, River Development and Ganga Rejuvenation, Government of India.
- DNRME. 2018. “Guideline for Failure Impacts Assessment of Water Dams”. Queensland Government, Australia.
- DEWS. 2017. “Guidelines of acceptable flood capacity for water dams”. Queensland Government, Australia.
- FEMA. 2004. “Federal Guidelines for Dam Safety – Glossary of Terms.” United States Federal Emergency Management Agency.
- FEMA. 2004. “Federal Guidelines for Dam Safety – Hazard Potential Classification System for Dams” United States Federal Emergency Management Agency.

- FEMA. 2012. “Assessing the Consequences of Dam Failure. A How-To Guide.” United States Federal Emergency Management Agency.
- FEMA. 2015. “Federal Guidelines for Dam Safety Risk Management.” United States Federal Emergency Management Agency.
- Fiedler, W.R. 2016. “Managing Dam Safety Risks related to Hydraulic Structures “, 6<sup>th</sup> International Symposium on Hydraulic Structures, IAHR, Portland, Oregon.
- Flint, Ed, et alia. 2019. “Structural Measures constraints in Maintaining the Health of DRP Dams “, International Dam Safety Conference, Ministry of Water Resources, River Development and Ganga Rejuvenation, Government of India.
- ICOLD. 1995. “Dam Failures – Statistical Analysis”. *Bulletin 99*. International Commission on Large Dams.
- ICOLD. 2015. “Flood Evaluation and Dam Safety “, Bulletin 170. International Commission on Large Dams.
- ICOLD. 2015. “Technical Advancement in Spillway Design “, Technical Committee on Hydraulics for Dams.
- ICOLD. 2005. “Dams and Flood Debris“, Committee on Blockage of Spillways and outlet Work, Bulletin. International Commission on Large Dams.
- ICOLD. 2005. “Uprating of Existing Dams “, General Reporter Q 90, 20<sup>o</sup> Large Dams Congress Brasilia, Brazil.
- ICOLD. 2005. “Dam Safety Management”, General Reporter Q91, 20<sup>o</sup> Large dams Congress, Brasilia, Brazil.
- ICOLD. 2019. “International Commission on Large Dams Webpage.” 2019. <http://www.icold-cigb.net/>.
- JCSS. 2008. “Risk Assessment in Engineering - Principle, System Representation and Risk Criteria”. Editor M.N. Faber.
- Khatsuria, R. M. 2005. “Hydraulics of Spillways and Energy Dissipators“, Editor Michael D. Meyer, New York.
- Marcano A., et alia. 1984. “Cavitation Damage at Chute 3 Stage 2 of Guri Spillway and Solution to the Problem”, International Conference on Incidents in Dams and Structures, Sao Paulo, Brazil.
- Mason P., et alia. 2003. “Ageing of dam components and dam safety regimes”. In XXI ICOLD, Q82, R.35, Vol. II, 599, Montreal
- SEAC, 2001. “Guía Técnica para La Elaboración de Planes de Emergencia de Presas”. Secretaria de Estado para Aguas y Costas. Ministerio del Medio Ambiente. España.
- SPANCOLD. 1997. “Aliviaderos y Descargas de Fondo”.
- SPANCOLD. 2012. “Risk Analysis as Applied to Dam Safety. Technical Guide on Operation of Dams and Reservoirs”. Vol. 1. Madrid: Professional Association of Civil Engineers. Spanish National Committee on Large Dams.
- Suárez, V. L, et alia. 2016. “Lecciones aprendidas de los Incidentes y fallas en la Presas de Venezuela“
- USACE. 2014. “Safety of Dams - Policy and Procedures. Engineering and Design. United States Army Corps of Engineers.” Washington D.C.: United States Army Corps of Engineers.

USBR. 1997. “Guidelines for Achieving Public Protection in Dam Safety Decision Making”. U.S. Department of the Interior, Bureau of Reclamation.

USBR. 2018. “Glossary of Terms”. U.S. Department of the Interior, Bureau of Reclamation. <http://www.usbr.gov/>.

USBR. 2001. “Risk Based Profiling System.” Denver: U. S. Bureau of Reclamation Technical Service Center.

World Bank Group. 2019. International Dam Safety Conference. Vol. II Compendium of Technical Papers, Ministry of Water Resources, River Development and Ganga Rejuvenation, Government of India. Bhubaneswar, Odisha, India.

## Chapter 2

BIS. “Code of practice for control of sediments in reservoirs”. Bureau of Indian Standards (BIS), – IS 6518.

BIS. “Fixing the Capacities of Reservoirs – Methods Dead Storage”. Bureau of Indian Standards (BIS), – IS 5477.

BIS. “Fixing the Capacities of Reservoirs – Methods General Requirements”. Bureau of Indian Standards (BIS), – IS 5477.

BIS. “Freeboard requirements in embankment Dams”. Bureau of Indian Standards (BIS), – IS 10635.

BIS. “Glossary of terms relating to river valley projects – Reservoirs”. Bureau of Indian Standards (BIS), – IS 4410.

BIS. “Guidelines for De-icing System for hydraulic installations”. Bureau of Indian Standards (BIS), – IS 10021.

BIS. “Guidelines for inspections and maintenance of dam and appurtenant structures”. Bureau of Indian Standards (BIS), – IS 9296.

BIS. “Methods for fixing the capacities of reservoir – Flood Storage”. Bureau of Indian Standards (BIS), – IS 5477.

BIS. “Methods for fixing the capacities of reservoir – Live Storage”. Bureau of Indian Standards (BIS), – IS 5477.

BIS. “Protection of slope for reservoir embankments”. Bureau of Indian Standards (BIS), – IS 8237.

Bureau of Reclamation, Veesaert, C.J. 2007. “Inspection of Spillways, Outlet works and Mechanical Equipment”. National Dam Safety Program Technical Seminar. Session XVI.

ICOLD – 2016. Technical Committee on Hydraulics for Dams, “Technical Advancements in Spillway Design”. Progress and Innovations from 1985-2015.

Swiss Committee on Dams. 2017. “Floating Debris at Reservoir Dam Spillways” Report of the on the state of floating debris issues at dam spillways Working group on floating debris at dam spillways.

Us Bureau of Reclamation, 1992, “Investigation on Debris and Safety Boom Alternatives for Bureau of Reclamation Use”

US Department of the Interior, 1992. “Investigation of Debris and Safety Booms Alternatives for Bureau of Reclamation Use”. US Bureau of Reclamation, Denver Office, Hydraulics Branch

US Department of the Interior, 1992. “Trash Control Structures and Equipment,-A Literature Review, US Bureau of Reclamation, Denver Office, Hydraulics Branch.

Wall, T., 1992. “Trash Controls Structures and Equipment, a literature review and survey of Bureau of Reclamation Experience”. US Department of the Interior

Wallerstein et alia. 1997. “Debris Control at Hydraulic Structures in Selected Areas of the United States and Europe” US Corps of Engineers

Wallerstein, N., et alia 1997. “Debris Control at Hydraulic Structures in Selected Areas of the United States and Europe”. USACE. Report CHL-CR-97-4.

Wark, B, et alia. 2015. Committee on Blockage of Spillways and Outlet Works

Department for the Environment, Food and Affairs. 2019. “Blockage Management Guide”, Report –SC 11 000 5/R2, Environment Agency, Wales Government

### Chapter 3

Afshar, A., et alia. 2012. “Fuse gates selection and operation: simulation–optimization approach”. Journal of Hydro informatics.

Alves, G., et alia. 2012. “Abrasive Effects Observed in Concrete Hydraulic Surfaces of Dams and Application of Repair Materials”, Abrasion Resistance of Materials, Dr. Marcin Adamiak (Ed.)

BIS. “Criteria for structural design of spillways pier and crest”. Bureau of Indian Standards (BIS), – IS 12720.

BIS. “Criteria for structural design of training walls and divide walls”. Bureau of Indian Standards (BIS), – IS 13551.

BIS. “Design of chute and side channel spillways - Criteria”. Bureau of Indian Standards (BIS), – IS 5186.

BIS. “Guidelines for Fixing Spillways Capacity”. Bureau of Indian Standards (BIS), – IS 11223.

BIS. “Guidelines for hydraulics design of barrages and weirs – Alluvial Reaches”. Bureau of Indian Standards (BIS), – IS 6966.

BIS. “Guidelines for operations and maintenance of barrages and weirs”. Bureau of Indian Standards (BIS), – IS 7349.

BIS. “Recommendations for hydraulics design of high ogge overflow spillways”. Bureau of Indian Standards (BIS), – IS 6934.

BIS. “Recommendations for inspections, testing and maintenance of radial gates and rope drum Hoist – Part 3”. Bureau of Indian Standards (BIS), – IS 10096.

Boes, R., Hager, W.; Two-Phase Flow Characteristics of Stepped Spillways; Journal of Hydraulic Engineering; ASCE; 2003.

Bureau of Reclamation, 1982. “Hydraulic Model Study of Ute Dam Labyrinth Spillway”. ERC.

Bureau of Reclamation, 2008. “Stability Analysis of Proposed Unlined Spillway Channel for Upper San Joaquin River Basin RM 274 Embankment Dam Alternative”. HILSG.

Bureau of Reclamation, 2014. “Appurtenant Structures for Dams (Spillways and Outlet Works)” Design Standard, Chapter 2: Hydrological Considerations.

- Bureau of Reclamation, 2014. “Appurtenant Structures for Dams (Spillways and Outlet Works)” Design Standard, Chapter 3: General Spillway Design Considerations.
- Bureau of Reclamation, Fritzell, K. et al. 2007. “Uplift and Crack Flow Resulting from Discharges over Open Offset Joints”. Laboratory Studies, DSO-07-07.
- Bureau of Reclamation, Trojanowski, J. 2006. “Can your spillway survive the next flood?”. Water Operation Maintenance Bulletin”, No. 218.
- Bureau of Reclamation, Veesaert, C.J. 2007. “Inspection of Spillways, Outlet works and Mechanical Equipment”. National Dam Safety Program Technical Seminar. Session XVI.
- Chanson, H.; Bung, D.B., Matos, J.; Stepped spillways and cascades; in "Energy Dissipation in
- Chanson, H.; Embankment Overflow Protection Systems and Earth Spillways; in Dams: Impacts, Stability and Design; Nova Science Publishers, Inc.; 2009
- Chanson, H.; Hydraulic Design of Stepped Spillways and Downstream Energy Dissipators; Dam Engineering; Vol XI; Issue 4
- Chanson, H.; The Hydraulics of Stepped Chutes and Spillways; A. A. Balkema Publishers;
- Chavan, B.; Hydraulic Design of Spillway and Aerator; E-proceedings of the 38th IAHR World Congress; Panama City, Panama; 2019
- Coleman, H. Wey, CY. 1985. “Hydraulic Design of Strontia Springs Diversion Dam, Colorado”. ASCE.
- Crookston, B. Tullis, B. Hydraulic Design and Analysis of Labyrinth Weirs. I: Discharge Relationships. ASCE.
- CWC. 2017. “National Register of Dams”, Central Water Commission, Ministry of Water Resources, River Development and Ganga Rejuvenation, Government of India.
- CWC. 2018. “Manual for Assessing the Structural Safety of Dams, DRIP, Ministry of Water Resources, River Development and Ganga Rejuvenation, Government of India.
- CWC. 2019. “Guidelines for Safety Inspection of Dams”, DRIP, Central Water Commission, Ministry of Water Resources, River Development and Ganga Rejuvenation, Government of India.
- CWC. 2019. “Manual for Rehabilitation of Existing Dams”, DRIP, Central Water Commission, Ministry of Water Resources, River Development and Ganga Rejuvenation, Government of India.
- Debris-Blocking Sensitivity of Piano Key Weirs under Reservoir-Type Approach Flow”. ASCE.
- Ebner, L., Fortuny, W., Hamernik, D., Hess, M., Sawka, M. 2016. USACE Portland District Spillway Gate Rehabilitation Program. Hydraulic Structures and Water System Management. 6th IAHR International Symposium on Hydraulic Structures, Portland, USA. B. Crookston & B. Tullis
- Erbisti, P. 214. Hydraulic Design of Gates. CRC Press.
- Epicum, S. Laugier, F. 2017. “Labyrinth and Piano Keys Weir III-PKW 2017”. Third International Workshop on Labyrinth and Piano Keys Weirs-2017. Vietnam.
- Fadaei Kermani, E. Barani, G.A. Ghaeini - Hessaroeyeh, M. 2013. “Investigation of Cavitation Damage Levels on Spillways”. World Applied Sciences Journal 21.
- Falvey, T., Treille, P. 1995. Hydraulic Design of Fuse Gates. Journal of Hydraulic Engineering, Vol. 121, No. 7, July, 1995. ASCE.

- FEMA. 2003. The National Dam Safety Program “Research Needs Workshop: Dam Spillways”. United States Federal Emergency Management Agency.
- FEMA. 2014. “Technical Manual: Overtopping Protection for Dams”. United States Federal Emergency Management Agency.
- Fiedler, W. 2016. “Managing Dam Safety Risks Related to Hydraulic Structures”. Hydraulic Structures and Water System Management. 6th IAHR International Symposium on Hydraulic Structures, Portland. B. Crookston & B. Tullis
- Frizell, K.W., Svoboda, C., Matos, J.; Performance of Type III Stilling Basins for Stepped Spillways; 2nd International Seminar on Dam Protection against Overtopping; Colorado, USA, 2016
- Gannett Fleming. 2019. Pikes Creek Dam Rehabilitation, Pennsylvania, USA.
- González, C., Chanson, H.; Flow Resistance and Design Guidelines for Embankment Stepped Chutes; Dams and Reservoirs, Societies and Environment in the 21st Century - Berga et al (eds)
- González, C., Chanson, H.; Hydraulic Design of Stepped Spillways and Downstream Energy Dissipators for Embankment Dams; Dam Engineering, Vol. XXVII, Issue 4
- Goodwin, C., at alia, ( 2019), “ Floating Bulkheads A Practical, Flexible and Economic approach to dewatering gates “ : Hydropower&Dams, Issue 3, 2019
- Guillaume, M. 2013. “PK Weir – design of a new economic shape of weir” HydroCoop.
- Guillaume, M. 2013. “The Piano Keys Weir: a new cost effective solution for spillways” HydroCoop.
- Guillaume, M. 2014. “Concrete Fuse plugs” ICOLD Bulletin on Cost saving in da Schleiss, Appendix 4.3. HydroCoop.
- Hansen, K. Fitzgerald, T. 2016. Performance of RCC Used for Overtopping Protection and Spillways. 2nd International Seminar on Dam Protection against Overtopping. Fort Collins. Colorado. USA.
- Hepler1, T. Young, J. Crookston, B. Crowder, J. 2018. “Successful Overtopping Protection Projects in the Eastern U.S.”. 3rd Int. Conference on Protection against Overtopping, UK.
- Hydraulic Structures”; IAHR Monograph, CRC Press, Taylor & Francis Group, 2015.
- ICOLD. 2005. “Report on save passage of extreme floods“, Technical Committee on Hydraulics for Dams.
- ICOLD. 2015. “Piano Keys Weirs Spillways“, Technical Committee on Hydraulics for Dams.
- ICOLD. 2016. “Technical Advances in Spillways Design, Progress and Innovations from 1985 to 2015”. Technical Committee on Hydraulics for Dams.
- Independent Forensic Team Report. 2018. “Oroville Dam Spillway Incident”.
- Jothiprakash, V., Bhosekar, V., Deolalikar, P.; Flow characteristics of orifice spillway aerator: Numerical model studies; ISH Journal of Hydraulic Engineering; 2015
- Khatsuria, R. M. 2005. “Hydraulics of Spillways and Energy Dissipators“. Editor Michael D. Meyer, New York.
- Khatsuria, R. M. 2008. “Remodeling of existing spillways-Part 1 and Part 2”. Hydrotopics.
- Koskinas, A. 2019. “Insights into the Oroville Dam 2017 Spillway Incident”. Geosciences.

- Lamperiere, F. 2013. “Collapsible gates and Labyrinth Weirs for new or existing dams”. Hydrocoop. France.
- Laugier, F. Gille, C. Ribeiro, M. Boillat, J. 2009. “Design and Construction of a Labyrinth PKW spillway at St. Marc Dam, France”. Hydropower & Dams.
- Loisel, P. Valley, P. Laugier, F. 2013. “Hydraulic physical model of Piano Keys Weir as additional flodd spillway on the Charmine Dam”. Labyrinth and Piano Keys Weirs II PKW 2013. Erpicum et alia.
- Lux, F., et alia, (2010) “ Simply Brilliant, New Approach for isolating Spillway gates “, Proceedings ASDSO, 2010
- Matos, J., Meireles, I.; Hydraulics of stepped weirs and dam spillways: engineering challenges, labyrinths of research; 5th International Symposium on Hydraulic Structures Brisbane, Australia, 2014
- Matos, J.; Sánchez-Juny, M., Quintela, A.; Dolz, J. Characteristic depth and pressure profiles in skimming flow over stepped spillways; 28th IAHR Congress. Graz; Austria; 1999
- Moreno, B., et alia ( 2006), “Repairing concrete structures at 95m water depth at Simon Bolivar (Guri) Dam “, “Recent Developments on Hydraulic Structures, International Symposium on Hydraulic Structures, Ciudad Guayana, Venezuela, A Marcano, Editor
- Ohtsu, I., Yasuda, Y., Takahashi, M.; Flow Characteristics of Skimming Flows in Stepped Channels; Journal of Hydraulic Engineering; ASCE; 2004.
- Patel, D., Patel, N., Hirpara, M., Parmar, R., Parekh, R.; Study of Fusegate on Wanakbori Weir; International Research Journal of Engineering and Technology; 2019
- Paxson,G. Crookston,B. Savage, B. Tullis,B. Lux III, F. The hydraulic design toolbox: Theory and Modelling for the Lake Townsend spillway replacement project
- Pfister, M. Capobianco, D. Tullis, B. Schleiss, A. 2013. “Debris-Blocking Sensitivity of Piano Key Weirs under Reservoir-Type Approach Flow”. ASCE.
- Pfister, M. Schleiss, A. 2013. “Effect of driftwood on hydraulic head of Piano Key weirs”.
- Pfister, M., Hager, W., Minor, H.; Bottom Aeration of Stepped Spillways; Journal of Hydraulic Engineering; ASCE; 2006.
- Ribeiro, M. Pfister, M. Schleiss, A. Boillat, J. 2012. “Hydraulic design of A-type Piano Key Weir”. Journal of Hydraulics Research. IAHEER.
- Saxena, K.R., Sharma, V.M. 2005. “Dams, Incidents and Accidents”. A.A. Balkema Publishers, India.
- Schleiss, A. 2011. “From Labyrinth to Piano Keys Weirs-A historical Review”. Labyrinth and Piano Keys Weirs – PKW 2011. Taylor & Francis Group. London.
- Schmocker, L. Höck, E. Mayor, P. Weitbrecht, V. 2014. “Hydraulic Model Study of the Fuse Plug Spillway at Hagneck Canal, Switzerland”.
- Simões, A.L., Schulz, H., Lobosco, R., De Melo, R.; Stepped Spillways: Theoretical, Experimental and Numerical Studies; University of São Paulo; Brazil; 2012
- Solanki, P., Shrimali, N., Ghandi, H. 2014. “A Study of Fusegate System on Dhatarwadi Dam, Amreli, Gujarat. Journal of International Academic Research for Multidisciplinary.
- SPANCOLD, Comité Nacional Español de Grandes Presas. 1997. Guías Técnicas de Seguridad de Presas, “Aliviaderos y Desagues”.

- Tiwari, H. Sharma, N. 2015. “Developments to improve hydraulic competence of spillways” International Conference on Water Re-Engineering on Water Resources and Ocean Engineering.
- US Department of the Interior, Bureau of Reclamation. 2014, “Appurtenant Structures for Dams – (Spillways and Outlet Works)- Design Standard, Chapter 3, Spillway design Considerations, Us Bureau of Reclamation.
- US Department of the Interior, Bureau of Reclamation. 2018, “Design Standard N°6, Hydraulic and Mechanical Equipment, Chapter 6, Bulkheads and Stop logs.
- USACE. 2004. “They might be giants - Terminus dam's new Fusegates”. Water Power & Dam Construction.
- USACE-USBR. 2015.”Failure of Radial (Tainter) Gates under Normal Operational Conditions”.
- USBR; Hydraulic Model Study on Fuse Plug Embankments; ERC-85-7; 1985
- USBR; Research State of the Art and Needs for Hydraulic Design of Stepped Spillways; HL 2005 06; July 2006.
- Wayne, H., Wei, C.I., Lindell, J.; Hydraulic Design of Spillways; Chapter 7; Hydraulic Design Handbook; 2004
- Wilhelms, S. Gulliver, J. 2005.”Gas Transfer, cavitation and bulking in self-aerated spillway flow”. Journal of Hydraulic Research Vol. 43, No. 5.
- Yildiz, D., Kas, I.; Hydraulic performance of stepped chute spillways; International Journal on Hydropower and Dams; 1998

## Chapter 4

- ANCOLD. 2015. “Flood Evaluation and Dam Safety”. Australian National Committee on Large Dams. Hobart, Australia.
- BIS. “Canal head regulators – Criteria for design”. Bureau of Indian Standards (BIS), – IS 6531.
- BIS. “Code of practice for design of tunnels conveying water – General design”. Bureau of Indian Standards (BIS), – IS 4880.
- BIS. “Code of practice for design of tunnels conveying water – Geometric design”. Bureau of Indian Standards (BIS), – IS 4880.
- BIS. “Code of practice for design of tunnels conveying water – Hydraulic design”. Bureau of Indian Standards (BIS), – IS 4880.
- BIS. “Criteria for hydraulic design of sluices in concrete and masonry dams”. Bureau of Indian Standards (BIS), – IS 11485.
- BIS. “Hydropower intakes – Criteria for hydraulic Design”. Bureau of Indian Standards (BIS), – IS 9761.
- BIS. 1977.“Guidelines for Design of Under-Seepage Control Measure for Earth and Rockfill Dams”. Bureau of Indian Standards (BIS), – IS 8414-1999, Govt. of India, 19 pages
- BIS. 1978. “Guidelines for Design of Large Earth and Rockfill Dams”. Bureau of Indian Standards (BIS), – IS 8414-1999, Govt. of India, 26 pages
- BIS. 1981.“Glossary of terms relating to river valley projects. Part XVI Gates and Valves. Section 2 Valves”. Bureau of Indian Standards (BIS), – IS 4410 (Part 16/2)-2001, Govt. of India, 17 pages

- BIS. 1985. “Criteria for hydraulic design of irrigation intake structures”. Bureau of Indian Standards (BIS), – IS 11570-1985, Govt. of India, 15 pages
- BIS. 1985. “Guidelines for Fixing Spillway Capacity”. Bureau of Indian Standards (BIS), – IS 11223-1985, Govt. of India, 10 pages
- BIS. 1985. “Hydraulic Design of Sluices in Concrete and Masonry Dams”. Bureau of Indian Standards (BIS), – IS 11485-1985, Govt. of India, 14 pages
- BIS. 1986. “Criteria for hydraulic design of Penstocks”. Bureau of Indian Standards (BIS), – IS 11625-1986, Govt. of India, 15 pages
- BIS. 1989. “Criteria for Estimation of Aeration Demand for Spillway and Outlet Structures”. Bureau of Indian Standards (BIS), – IS 12804-2004, Govt. of India, 14 pages
- BIS. 1992. “Code of Practice for Control of Sediment in Reservoirs”. Bureau of Indian Standards (BIS), – IS 6518-1998, Govt. of India, 7 pages
- BIS. 1992. “Glossary of terms relating to river valley projects. Part VIII Dams and Dam Sections”. Bureau of Indian Standards (BIS), – IS 4410 (Part 8)-1992, Govt. of India, 24 pages
- BIS. 1995. “Hydropower Intakes - Criteria for hydraulic design”. Bureau of Indian Standards (BIS), – IS 9761-1995, Govt. of India, 31 pages
- BIS. 1995. “Recommendations for Design of Trash Racks for Intakes”. Bureau of Indian Standards (BIS), – IS 11388-1995, Govt. of India, 6 pages
- BIS. 1999. “Glossary of terms relating to river valley projects. Part XVI Gates and Valves. Section 1 Gates and Terms Related with Gates”. Bureau of Indian Standards (BIS), – IS 4410 (Part 16/1)-2004, Govt. of India, 39 pages
- BIS. 2003. “Hydrometric Determinations – Vocabulary and Symbols”. Bureau of Indian Standards (BIS), – IS 1191-2003, Govt. of India, 41 pages
- Bureau of Reclamation, Veesaert, C.J. 2007. “Inspection of Spillways, Outlet works and Mechanical Equipment”. National Dam Safety Program Technical Seminar. Session XVI.
- Calamak, M., et alia. 2018. “A Review of the Anita Dam Incident: Internal Erosion caused by a buried conduit and Lesson learned”, 5th International Symposium on Dam Safety, Istanbul, Turkey.
- Carrillo, J.M. 2009. “Evaluación de la Seguridad Hidrológico-Hidráulica de la Presa de Compuerta (T.M. de Velilla del río Carrión, Provincia de Palencia) perteneciente a la Cuenca Hidrográfica del Duero”. Universidad Politécnica de Valencia, España.
- CDA. 2007. “Dam Safety Guidelines”. Canadian Dam Association, ACB.
- CWC. 2017. “National Register of Dams”, Central Water Commission, Ministry of Water Resources, River Development and Ganga Rejuvenation, Government of India.
- CWC. 2018. “Handbook for Assessing and managing Reservoir Sedimentation”, DRIP, Central Water Commission, Ministry of Water Resources, River Development and Ganga Rejuvenation, Government of India.
- CWC. 2018. “Manual for Assessing the Structural Safety of Dams, DRIP, Ministry of Water Resources, River Development and Ganga Rejuvenation, Government of India.
- CWC. 2019. “Guidelines for Assessing and managing Risks associated with Dams”, DRIP, Central Water Commission, Ministry

- CWC. 2019. “Guidelines for Safety Inspection of Dams”, DRIP, Central Water Commission, Ministry of Water Resources, River Development and Ganga Rejuvenation, Government of India.
- CWC. 2019. “Manual for Rehabilitation of Existing Dams”, DRIP, Central Water Commission, Ministry of Water Resources, River Development and Ganga Rejuvenation, Government of India.
- Department for the Environment, Food and Affairs. 2019. “Blockage Management Guide”, Report –SC 11 000 5/R2, Environment Agency, Wales Government
- Falvey, H. 1990. “Cavitation in Chutes and Spillways”. Engineering Monograph N° 42. A Water Resources Technical Publication. U.S. Department of the Interior. Bureau of Reclamation. 145 pages.
- FEMA. 2000. “The National Dam Safety Program. Research Needs Workshop: Seepage through Embankment Dams” United States Federal Emergency Management Agency.
- FEMA. 2004. “Federal Guidelines for Dam Safety – Glossary of Terms.” United States Federal Emergency Management Agency.
- FEMA. 2004. “The National Dam Safety Program. Research Needs Workshop: Outlet Works” United States Federal Emergency Management Agency.
- FEMA. 2005. “Technical Manual: Conduits through Embankment Dams. Best practices for Design, Construction, Problem Identification and Evaluation, Inspection, Maintenance, Renovation and Repair.” United States Federal Emergency Management Agency.
- FEMA. 2010. “Technical Manual: Outlet Works Energy Dissipators. Best practices for Design, Construction, Problem Identification and Evaluation, Inspection, Maintenance, Renovation and Repair.” United States Federal Emergency Management Agency.
- FEMA. 2015. “Federal Guidelines for Dam Safety Risk Management.” United States Federal Emergency Management Agency.
- Fiedler, W.R. 2016. “Managing Dam Safety Risks related to Hydraulic Structures “, 6th International Symposium on Hydraulic Structures, IAHR, Portland, Oregon.
- Fischer, Gary. "Cured-In-Place Pipe - A Cure for Outlet Problems." ASDSO West Regional Conference. 2009.
- Flint, Ed, et alia. 2019. “Structural Measures constraints in Maintaining the Health of DRP Dams”, International Dam Safety Conference, Ministry of Water Resources, River Development and Ganga Rejuvenation, Government of India.
- Foster, M. et alia. 2000. “The statistics of embankment dam failures and accidents” Canadian Geotechnical Journal. Volume 37 Number 5, pp 1000-1024
- Hidrotec Ingenieros. 1991. “Actualización Parámetros Hidráulicos. Sistema Hidráulico Turimiquire”. Ministerio del Ambiente.
- ICOLD. 1995. “Dam Failures – Statistical Analysis”. Bulletin 99. International Commission on Large Dams.
- ICOLD. 2005. “Dam Safety Management”, General Reporter Q91, 20° Large dams Congress, Brasilia, Brazil.
- ICOLD. 2005. “Dams and Flood Debris“, Committee on Blockage of Spillways and outlet Work, Bulletin. International Commission on Large Dams.

- ICOLD. 2005. “Upgrading of Existing Dams”, General Reporter Q 90, 20<sup>o</sup> Large Dams Congress Brasilia, Brazil.
- ICOLD. 2019. “International Commission on Large Dams Webpage.” 2019. <http://www.icold-cigb.net/>.
- Kosak, J. et alia. 2016. “Transition of cavitating flow to supercavitation within Venturi nozzle – hysteresis investigation”, EPJ Conference 143 – Experimental Fluids Mechanics..
- Mason P., et alia. 2003. “Ageing of dam components and dam safety regimes”. In XXI ICOLD, Q82, R.35, Vol. II, 599, Montreal
- Möller, G. 2013. “Vortex-Induced Air Entrainment Rate at Intakes”, Versuchsanst. f. Wasserbau, Hydrologie u. Glaziologie, Zurich. 223 pages.
- of Water Resources, River Development and Ganga Rejuvenation, Government of India.
- SEAC, 2001. “Guía Técnica para La Elaboración de Planes de Emergencia de Presas”. Secretaria de Estado para Aguas y Costas. Ministerio del Medio Ambiente. España.
- Shuai, L. et alia. 2016. “Evolution of Pressure and Cavitation on Side Walls Affected by Lateral Divergence Angle and Opening of Radial Gate”. Journal of Hydraulic Engineering. ASCE.
- SPANCOLD. 1997. “Aliviaderos y Desgües”. Guía Técnicas de Seguridad de Presas. Guía N<sup>o</sup> 5. Comité Nacional de Grandes Presas, Madrid.
- SPANCOLD. 2012. “Risk Analysis as Applied to Dam Safety. Technical Guide on Operation of Dams and Reservoirs”. Vol. 1. Madrid: Professional Association of Civil Engineers. Spanish National Committee on Large Dams.
- US Department of the Interior, 1992. “Trash Control Structures and Equipment,-A Literature Review, US Bureau of Reclamation, Denver Office, Hydraulics Branch.
- USACE. 1995. “Guidelines for trenchless technology : Cured-In-Place Pipe (CIPP), Fold-and-Formed Pipe (FFP), Mini-Horizontal Directional Drilling (Mini-HDD) and Microtunneling”. United States Army Corps of Engineers. Washington D.C.: United States Army Corps of Engineers.
- USACE. 2014. “Safety of Dams - Policy and Procedures. Engineering and Design. United States Army Corps of Engineers.” Washington D.C.: United States Army Corps of Engineers.
- USBR. 2005. “Spillway, Outlet Works, and Water Conveyance Structure Needs – Survey Results”. U.S. Department of the Interior, Bureau of Reclamation.
- USBR. 2018. “Glossary of Terms”. U.S. Department of the Interior, Bureau of Reclamation. <http://www.usbr.gov/library/glossary>.
- USDA. 1990. “Earth Dams and Reservoirs”. U.S. Department of Agriculture Soil Conservation Service. Engineering Division. Technical Release N<sup>o</sup> 60.
- USDA. 1994. “Gradation Design of Sand and Gravel Filters”. Natural Resources Conservation Service. Natural Engineering Handbook. Part 633, Chapter 26. U.S. Department of Agriculture.
- USDA. 2007. “Filter Diaphragms”. Natural Resources Conservation Service. Natural Engineering Handbook. Part 628, Chapter 45. U.S. Department of Agriculture.
- Wall, T., 1992. “Trash Controls Structures and Equipment, a literature review and survey of Bureau of Reclamation Experience”. US Department of the Interior
- Wang, X. et alia. 2019. “Experimental Study on the Mechanism of the Combined Action of Cavitation Erosion and Abrasion at High Speed Flow”. International Journal of Concrete Structures and Materials. Mater 13, 58.

Wark, B, et alia. 2015. Committee on Blockage of Spillways and Outlet Works

World Bank Group. 2019. International Dam Safety Conference. Vol. II Compendium of Technical Papers, Ministry of Water Resources, River Development and Ganga Rejuvenation, Government of India. Bhubaneswar, Odisha, India.

Zhang, L. et alia. 2009. “Analysis of earth dam failures: A database approach”. *Georisk: Assessment and Management of Risk for Engineered Systems and Geohazards*, 3:3, 184-189.

## Chapter 5

AASDO, 2000, “ Spillway gate Workshop”, Epri

American Concrete Institute. 2017. “Report on the Erosion of Concrete in Hydraulic Structures”. ACI 207.6R-17.

Binnie and Partners, “ Mangla Dam project “, Completion report, Vol III, River Control, 1971

BIS. “Criteria for estimation of aeration demand for spillway & outlet works”. Bureau of Indian Standards (BIS), – IS 12804.

BIS. “Criteria for hydraulic design of buckets type Energy Dissipators”. Bureau of Indian Standards (BIS), – IS 7365.

BIS. “Criteria for hydraulic jump type stilling basing with horizontal and sloping apron”. Bureau of Indian Standards (BIS), – IS 4997.

BIS. “Criteria for structural design of energy dissipators for spillways”. Bureau of Indian Standards (BIS), – IS 11527.

BIS. “Guidelines for design of drainage arrangements of energy dissipators and training walls of spillways”. Bureau of Indian Standards (BIS), – IS 11772.

BIS. “Guidelines for selection of spillways and energy dissipator”. Bureau of Indian Standards (BIS), – IS 10137.

BIS. “Hydraulic Design of Stilling Basing”. Bureau of Indian Standards (BIS), – IS 4997.

BIS. “Preliminary design, operation and maintenance of protection works downstream of spillways - Guidelines”. Bureau of Indian Standards (BIS), – IS 13195.

Bollaert, E. Lesleighter, E. 2014. “Spillway Rock Scour Experience and Analysis - the Australian Scene over the past Four Decades”. 5th International Symposium on Hydraulic Structures. HSS: ECE. Australia.

Bollaert, E. Lesleighter, E. et al. 2016. “Rock scour in Australia: Some latest Queensland Experiences”. *Scour & Erosion. Proceedings of the 8<sup>th</sup> International Conference on Scour and Erosion.*

Bureau of Reclamation, 2014. “Appurtenant Structures for Dams (Spillways and Outlet Works)” Design Standard, Chapter 3: General Spillway Design Considerations.

Bureau of Reclamation, 2019. “Flow deflectors to reduce abrasion damage in stilling basins, 40 years progress report”. Hydraulic Laboratory Report HL-2019-02.

Bureau of Reclamation, Veesaert, C.J. 2007. “Inspection of Spillways, Outlet works and Mechanical Equipment”. National Dam Safety Program Technical Seminar. Session XVI.

- Burgi P.H. et al. (1988). "Spillway Performance and Remedial Measures". In: Jansen R.B. (eds) Advanced Dam Engineering for Design, Construction, and Rehabilitation. Springer, Boston, MA.
- Burn, D.H., et al, "Hydraulic Model Analysis for an Orifice Spillway ", Canadian Journal of Civil Engineering, Vol 17, pp780-789, 1990
- Burn, D.H., et al; Hydraulic Model Analysis for an Orifice Spillway; Canadian Journal of Civil Engineering; Vol 17; 1990
- Castillejo, N., " Hydraulic Aspects of the Caruachi spillway and dynamic behavior of lower radial gate edge in partly drowned conditions " Mac thesis , IHE, Delft Hydraulics
- Castro C, (1992), " Estudio Teórico-Experimental de un Descargador de Fondo con Aliviadero de Servicio de Grandes Presas ", Trabajo de Grado presentado a la ilustre Universidad Central de Venezuela para optar al Título de Magister Scientiarum en Ingeniería Hidráulica, ( In Spanish) Caracas 1992
- Castro M., et al, "Estudio Experimental del Funcionamiento de Dispositivos aireado res en superficies horizontales de grandes presas ", Trabajo Especial de Grado , UCAB 1991 ( In spanish)
- Castro, C.; "Estudio Teórico-Experimental de un Descargador de Fondo con Aliviadero de Servicio de Grandes Presas "; TG, MSc, UCV, Caracas; Venezuela; 1992
- Chanson, H. 2015."Energy Dissipation in Hydraulic Structures". IAHR Monograph. CRC Press.
- Chen, Ju, " The problem of energy dissipation and Erosion by Sky jump for high dams ", international Symposium on Hydraulics for Large Dams, Beijing, China, 1988
- China National Committee Large Dams, (1979), 2 Studies on the Configuration of short intakes for free flow Spillways tunnels, Q 50, R587, XXIII Congress If Large Dams, New Delhi
- Choudhury, MR. Das, S. Nigam, U. 2015." Overview of energy dissipators and stilling basins with design aspects of hydraulic jump type energy dissipators". NCIET.
- CWC. 2017. "National Register of Dams", Central Water Commission, Ministry of Water Resources, River Development and Ganga Rejuvenation, Government of India.
- CWC. 2018. "Manual for Assessing the Structural Safety of Dams, DRIP, Ministry of Water Resources, River Development and Ganga Rejuvenation, Government of India.
- CWC. 2019. "Guidelines for Safety Inspection of Dams", DRIP, Central Water Commission, Ministry of Water Resources, River Development and Ganga Rejuvenation, Government of India.
- CWC. 2019. "Manual for Rehabilitation of Existing Dams", DRIP, Central Water Commission, Ministry of Water Resources, River Development and Ganga Rejuvenation, Government of India.
- CWC. 2019. "Rehabilitation and Improvement of Hirakud Dam including Additional Spillway under DRIP". Odisha.
- CWC. Undated." Maneri Dam - Repair of Spillway and its Energy dissipater severely damaged by rolling boulders during floods - Case study".
- Desai, V. Patil, A. Pillai, S. 2014."Repairs to the damage in the baffle blocks of Salauli Dam using PICC materials". Dam Engineering, Volume XXV, Issue 1.
- Falvey , H., " Prevention of cavitation on Chutes and spillways ", Proceedings of the Conference on Frontiers in Hydraulic Engineering", MIT, Cambridge, Massachusetts, 1983

- FEMA. 2003. The National Dam Safety Program “Research Needs Workshop: Dam Spillways”. United States Federal Emergency Management Agency.
- FEMA. 2010. “Technical Manual: Outlet works energy dissipators”. P-679. United States Federal Emergency Management Agency.
- Green, D. W., “ Chira Piura, a Project to improve irrigation in Northern Peru”, Water Power and Dam Construction, 1975
- Hager, W. 1992. “Energy Dissipators and Hydraulic Jump”. Laboratory of Hydraulics, ETH, Switzerland.
- Hasan, M. Li, S. Zsaki, A. Nokken, M. 2019.”Measurement of Abrasion on Concrete Surfaces with 3D Scanning Technology”. ASCE. J. Mater. Civ. Eng.
- ICOLD, (1986) “ Design of Spillways for Dams “-Bulletin
- ICOLD. 1987. “Spillways for dams”. Bulletin N°58.
- ICOLD. 2016. “Technical Advances in Spillways Design, Progress and Innovations from 1985 to 2015”. Technical Committee on Hydraulics for Dams.
- IS 11485.2004.”Criteria for Hydraulic Design of sluices y concrete and masonry dams”. Indian Standards Institute. New Delhi, India.
- IS 11527 (1985, (reaffirmed at 2004). “Criteria for structural design of energy dissipators for spillways”. Bureau of Indian Standards. New Delhi, India.
- IS 4897. 1995. “Criteria for design of hydraulic jump type stilling basins with horizontal and sloping apron”. Bureau of Indian Standards. New Delhi, India.
- IS 7365. 2010. “Criteria for hydraulic design of bucket type energy dissipators”. Bureau of Indian Standards. New Delhi, India.
- Khatsuria, R. M. 2005. “Hydraulics of Spillways and Energy Dissipators“. Editor Michael D. Meyer, New York.
- Khatsuria, R.M.( 2005), “ Hydraulics of Spillways and Energy Dissipation “, Marcel Dekker, ISBN: 0-8247-5789-0
- Mac Tavish, G., et alia, (1985) “The Jebba hydroelectric Project “, Water Power and Dam Construction
- Marcano a. et al , “Selection of the Energy Dissipaters for the spillway of Lower Caroni Projects “, Q 63, R45: XVI Congress on Large Adams, san Francisco, 1988
- Marcano, A. 2000. “Energy dissipators Bucket Type at Bajo Caroní HE Development”. EDELCA, Venezuela.
- McDonalds, J. USACE. 1980. “Maintenance and preservation of concrete structures, Repair of erosion damaged structures”. TR -78-4, WES.
- Neal Fraser, C. 2016. “Ski jump energy dissipation. Design of a ski jump to maximize energy dissipation and aeration”. Master of Engineering Thesis. University of Stellenbosch. South Africa.
- Novak, P Editor, “ Developments in Hydraulic Engineering.2”, Elsevier, 1984
- Prasanna, S. Suresh Kumar, N. 2019.” Simulation and Analytical Estimation of Spillway Flip Bucket Parameters”. IJITEE. ISSN: 2278-3075, Volume-8 Issue-12.

- Ramarao, V.S., Inamdar, A., Joshi, P., Sanghvi, S.; Assessment of ski-jump bucket parameters for sluice spillway of Himalayan dam by hydraulic model studies; International Journal of Advance Research, Ideas and Innovations in Technology; 2019.
- Richard, G. Schweiger, P. Israel, B. 2017. "What's happens downstream, stays downstream or does it?. Potential Failure Modes from spillways exit flow". ASDSO Inc.
- Saxena, K.R., Sharma, V.M. 2005. "Dams, Incidents and Accidents". A.A. Balkema Publishers, India.
- Schleiss, A. (no date). "Scour evaluation in space and time - the challenge of dam designers". LCH, Switzerland.
- Semenkov, V., "general reporter", Q 50, XIII Congress of Large Dams, New Delhi, 1979
- Shoutan, Z., "Latest developments in Hydraulic Design of Outlet Works in China", Bulletin N° Trita-vBI-154, Hydraulic Laboratory, The Royal Institute of Technology, Stockholm, Sweden, 1991
- Shoutan, Z., "Latest developments in Hydraulic Design of Outlet Works in China", Bulletin N° Trita-vBI-154, Hydraulic Laboratory, The Royal Institute of Technology, Stockholm, Sweden, 1991
- Sims, G.P. "The aging and rehabilitation of appurtenant structures to dams and the aging of masonry dams". Brown and Root Ltd., UK.
- Sobani, A. Merh, S.H. 2014. "Pressure fluctuations on the slabs of stilling basins under hydraulic jump". 11th International Conference on Hydroinformatics. New York.
- SPANCOLD, Comité Nacional Español de Grandes Presas. 1997. Guías Técnicas de Seguridad de Presas, "Aliviaderos y Desagues".
- US Corps of Engineers, (1977), "Hydraulic Design Criteria ", Dip of the Interior
- USACE. 1992. "Hydraulic Design of Spillways".EM-1110-2-1603, Washington.
- USACE. 1999. "Repair and rehabilitation of dams: Case studies".TR REMR-CS-83, Washington.
- USSD. 2017. "Improving Reliability of Commonly Used Hydraulic Valves". Committee on Hydraulics of Dams, US Society on Dams, Colorado.
- Viola. B. V., " et al, ( 1983), " Magat River multipurpose Project", Water Power and dam Construction, Dec 1983
- Wan, W. Liu, B. Raza, A. 2018. "Numerical Prediction and Risk Analysis of Hydraulic Cavitation Damage in a High-Speed-Flow Spillway". Hindawi, Shock and Vibration.
- Wei C, Y, et alia, [Potential Flow Analysis. Flo Pattern and velocity using Finite Elements ", Users manual. Hydraulic Engineering Department, Harza Eng Co, 1986
- Wei C.Y., (1993), In " Davis[ Handbook of Applied hydraulics ", Chapter 16, Ziparro Hasen Editors, Mc Graw Hill, Fourth Edition
- Wei, C Y, et al, " Simulation of free jet trajectories for the design of aeration devices on hydraulic Structures ", proceedings, 4<sup>th</sup> International Conference on Finite Elements , Hannover, 1982
- Zhuoyi, D., " Study on Hydraulic Characteristics of Bottom Outlets with High Head", International Symposium on Hydraulics for High Dams, Beijing, China, 1982

# Central Dam Safety Organisation

## Central Water Commission

### Vision

To remain as a premier organisation with best technical and managerial expertise for providing advisory services on matters relating to dam safety.

### Mission

To provide expert services to State Dam Safety Organisations, dam owners, dam operating agencies and others concerned for ensuring safe functioning of dams with a view to protect human life, property and the environment.

### Values

**Integrity:** Act with integrity and honesty in all our actions and practices.

**Commitment:** Ensure good working conditions for employees and encourage professional excellence.

**Transparency:** Ensure clear, accurate and complete information in communications with stakeholders and take all decisions openly based on reliable information.

**Quality of service:** Provide state-of-the-art technical and managerial services within agreed time frame.

**Striving towards excellence:** Promote continual improvement as an integral part of our working and strive towards excellence in all our endeavours.

### Quality Policy

We provide technical and managerial assistance to dam owners and State Dam Safety Organizations for proper surveillance, inspection, operation and maintenance of all dams and appurtenant works in India to ensure safe functioning of dams and protecting human life, property and the environment.

We develop and nurture competent manpower and equip ourselves with state of the art technical infrastructure to provide expert services to all stakeholders.

We continually improve our systems, processes and services to ensure satis-

