Manual for Assessing Hydraulic Safety of Existing Dams - Volume II

Doc. No. CDSO_MAN_DS_04_v1.0

June 2021

Dam Rehabilitation & Improvement Project

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.

सत्यमेव जयते

Government of India
Central Water Commission
Central Dam Safety Organization

Manual for Assessing Hydraulic Safety of Existing Dams
Volume II

June 2021

Dam Safety Rehabilitation Directorate
3rd Floor, New Library Building
R. K. Puram
New Delhi - 110066

Government of India
Central Water Commission
Central Dam Safety Organisation

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**.**

For any information, please contact:
The Director
Dam Safety Rehabilitation Directorate
Central Dam Safety Organisation
Central Water Commission
3rd Floor, New Library Building (Near Sewa Bhawan)
R. K. Puram, New Delhi – 110066.
Email: dir-drip-cwc@nic.in

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

This page has been left blank intentionally.

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.

(Dr. R K Gupta)
Member (D&R)
Central Water Commission

New Delhi
June 2021

This page has been left blank intentionally.

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, Venezuala 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, Venezuala 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.

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.

This page has been left blank intentionally.

Manual Review Committee

Chairman	Gulshan Raj	Chief Engineer, Dam Safety Organization, Central Water Commission, New Delhi
Member Secretary	Pramod Narayan	Director, Dam Safety Rehabilitation Directorate and Project Director, DRIP, Central Water Commission, New Delhi

Committee Members

Vivek Tripathi	Director, Concrete & Masonry Dam Designs (East & North East), Central Water Commission, New Delhi
Zulfequar Ahmad	Professor, Civil Engineering, IIT Roorkee, Uttarakhand
K G Sharma	Professor, IIT Delhi
P B Deolalikar	Retd. Scientist, Central Water and Power Research Station, Pune, Maharashtra
Y K Bhadane	Superintending Engineer, DSO Nasik, Government of Maharashtra
Ajay Mittal	General Manager (Civil), NHPC, Faridabad, Haryana

Team involved in preparing this Manual

Pramod Narayan	Director, Dam Safety Rehabilitation Directorate and Project Director, DRIP, Central Water Commission, New Delhi
Saurabh Sharan	Deputy Director, Dam Safety Rehabilitation Directorate, Central Water Commission, New Delhi
Gaurav Singhai	Deputy Director, Dam Safety Rehabilitation Directorate, Central Water Commission, New Delhi
CPMU Consultant	

Contents

List of figures

ABBREVIATIONS

Acronyms used in this publication are as follows:	
ANCOLD	Australian National Committee on Large Dams
ASCE	American Society of Civil Engineers
BDS	British Dam Society
BIS	Bureau of Indian Standards
BRE	Building Research Establishment
BSI	British Standards Institution
CIRIA	Construction Industry Research and Information Association
CWC	Central Water Commission
D&R	Design & Research Wing
DDMA	District Disaster Management Authority
DRIP	Dam Rehabilitation and Improvement Project
DSRP	Dam Safety Review Panel
DSO	Dam Safety Organization
DVC	Damodar Valley Corporation
EDELCA	Electrificación del Caroní (Venezuela)
FEMA	Federal Emergency Management Agency (USA)
ICE	Institution of Civil Engineers
ICOLD	International Commission on Large Dams
IMD	Indian Meteorological Department
JWA	Japan Water Agency
MoWR	Ministry of Water Resources (India)
O&M	Operations and maintenance
PAR	Population at Risk
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
SBA	Standards Based Approach
SDSO	State Dam Safety Organization
SPANCOLD	Spanish Committee of Large Dams
USACE	U.S. Army Corps of Engineers
USBR	U.S. Bureau of Reclamation
USCOLD	U.S. Commission on Large Dams
USSD	U.S. Society on Dams

This page has been left blank intentionally.

APPENDIX A–FAILURE MODE IDENTIFICATION

Identification of Failure Modes and the conditions and events that may take place for failure to occur represents a qualitative part of the process of Risk Analysis, which is a first component of Risk Management process, considered by CWC, through DRIP Project by the "Guidelines for Assessing and Managing Risks Associated with Dams" (CWC, 2019).

Risk Management in a broad context, encompasses activities related to making Risk Informed decisions and prioritizing studies, surveys, instrumentation, flood studies update, to achieve risk reduction and Risk Mitigation actions (either structural or nonstructural, or both) and making program decisions associated with managing a portfolio of dams.

Identification of Failure Modes process is actually in the general context located at the heading of a comprehensive process of Risk Informed Decisions (Figure A-1) and it may be used to represent a future application as part of the initial task of handling a Risk Informed Decision Process for a portfolio of Dams in India.

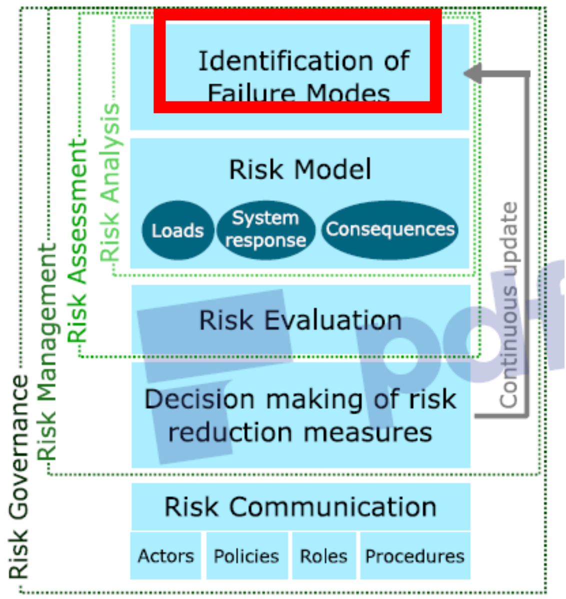


Figure A- 1: Components of a Risk Informed Decision Process (CWC, 2019)

Failure Mode (FM) is defined (CWC, 2019), as: "*a sequence of events that may cause failure or disrupt the function of the dam-reservoir system or part of it*". That Guideline also adds that this series of events are associated to a determined load scenario and to a logical sequence, which starts with a main initiating event, followed by a chain of development or propagation events and culminating either in a dam failure or, in a loss of operability of an appurtenant work;

FM identification is an important task related to dam safety evaluation in any phase of the life of a dam: design, construction, operation, surveillance, rehabilitation and, of course, within the process of Risk Analysis. Within the scope of the "Manual on Assessing Hydraulic Safety of Existing Dams" (MAHSED), FM Identification allows to understand and to balance physical and functional conditions of appurtenant works that could influence the hydraulic safety of dam-reservoir system, in order to proceed with adequate engineering actions and further, with rehabilitation measures.

A FM Identification encompasses all credible and physically possible process that could progress up to dam failure, resulting from an existing inadequacy or defect related to natural conditions (geology, hydrology), design, construction, materials, hydraulic functioning, operation and maintenance, and ageing. Any of (or sum of) these conditions may lead to an uncontrolled release of the water from the reservoir, or a temporal loss of water availability in the reservoir, unavailability of the water release structures, or other type of what can be considered a total or partial loss of operability of the project. The factors to be included in the potential failure process are grouped in:

(a) Those that trigger, favor or make more probable the occurrence of the failure process, named **"More likely"** or adverse factors and,

(b) Those that avoid or make less probable the occurrence of the process, named **"Less likely"** or favorable factors.

The process to identify a FM, a classification of FM and templates to be used for selecting the FM is included elsewhere (CWC, 2019). Based on those guidelines, Appendix A has included a series of generic and likely failure modes related to appurtenant works, within the context of MAHSED. The basic concepts adopted to select the FM are:

- **Scenarios:** extreme event and normal operation (any-day), related with spillway capacity and hydraulic malfunction; thus, the triggering events (hazards) are: floods and several hydraulic loads resulting from flow condition.
- **Existing dams:** whose hydraulic safety has to be improved due to ageing, updating or upgrading with rehabilitation measures.
- **Identification of a FM:** results from inspection, evaluation of hydraulic safety, surveillance and data analysis, or from a Risk Analysis.
- **Failure mechanism:** can be imminent, in progress, credible of developing (with available information) and credible of developing (but with lack of information). The type of engineering actuations and the rehabilitation measures will vary in each case.

Figure A-2 shows the sequence of a FM as considered in MAHSED, for Normal and Hydrological scenarios of loads. Failure could be dambreak or disruption of an appurtenance work.

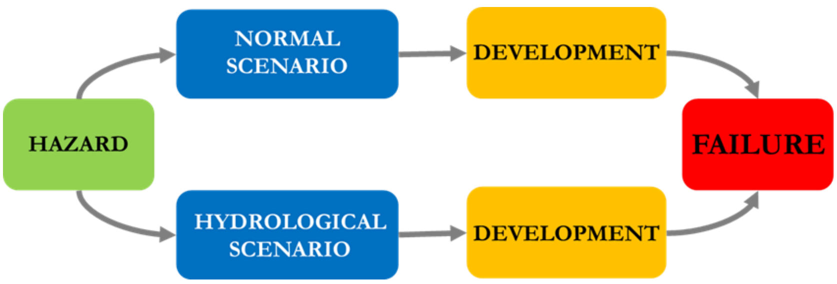


Figure A- 2: Failure modes sequence (Adapted IPresas, 2019)

The selected FM, in the MAHSED context, have to do with dam overtopping and structural damages with complete or temporal loss of operability of a hydraulic structure, due to:

1) Loss of function of spill capacity
2) Occurrence of hydraulic load acting during one event or, in continuous fashion.
3) Operative failure of valves and gates due to any cause: electricity disruption, mechanical failure, human error or obstruction (ice, floating debris).

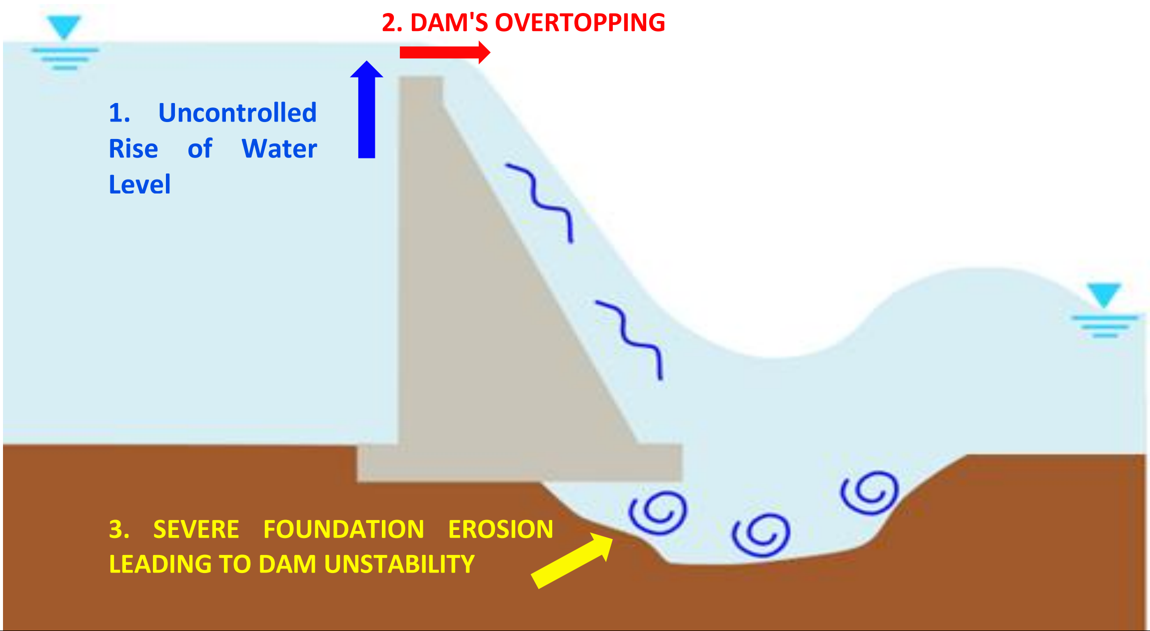
Other failures caused by vandalism, sabotage or earthquakes are addressed by other guidelines, so they are not included in this Appendix. It has to bear in mind that each dam-reservoir system has to be deeply analyzed to define specific and credible failure modes.

Table A-1 is a list of 23 recurrent failure modes in dams related to hydrological scenarios or caused by hydraulic and non-hydraulic problems but affecting hydraulics work operability. Following, these FM are presented and described using typical CWC template.

Failure Mode #	Description
SPILLWAY	
HYDROLOGICAL SCENARIO FLOOD	
Dam overtopping during extreme event → potential dambreak and uncontrolled release	
#1	Concrete dam overtopping due to insufficient capacity of spillway
#2	Embankment dam overtopping due to insufficient spillway capacity
#3	Dam overtopping caused by gate failure
#4	Spillway blockage by debris, gates jamming and dam overtopping
HYDROLOGICAL OR NORMAL SCENARIO	
Disruption of spillway function: damage/collapse of a component →loss of hydraulic safety	
#5	Chute wall overtopping in ungated spillway
#6	Cavitation damage in chute and energy dissipators
#7	Hydrodynamic pressure (hydraulic jacking) in chute or energy dissipators
#8	Erosion of chute foundation
#9	Cavitation damage downstream of an aeration device
#10	Malfunction of spillway flip bucket due to partial submergence of the jet
#11	Malfunction of stilling basin
#12	Unstable hydraulic jump in the stilling basin
#13	Ungated spillway toe erosion due to weir flow at the bucket lip
#14	Plunge pool erosion
#15	Bank erosion due to poor flow exit
OUTLET WORKS	
NORMAL SCENARIO	
Disruption of outlet function: damage/collapse of a component →loss of hydraulic safety	
#16	Bottom outlet blockage
#17	Intake structure blockage
#18	Erosion at Outlet caused by inadequate Energy Dissipator
#19	Piping (or internal erosion) on the embankment due to filtration from pressurized conduit
#20	Internal erosion in embankment dam due to flow into a non-pressurized conduit
#21	Internal erosion in embankment due to structural failure of conduit
HYDROMECHANICAL EQUIPMENT: SPILLWAY OR OUTLET WORKS	
HYDROLOGICAL OR NORMAL SCENARIO	
#22	Gates and/or valves jamming
#23	Malfunction of sluice gate operation at partial gate openings

Table A-1 Description of generic failure modes

Failure Mode 1	CONCRETE/MASONRY DAM OVERTOPPING DUE TO INSUFFICIENT CAPACITY OF SPILLWAY
Description	
The spillway has insufficient discharging capacity to pass the design flood or other extreme floods and inadequate safety free-board. Reservoir level overcomes dam freeboard and water passes over the crest. Flow on downstream face reaches the toe and causes backwards bed erosion and dam instability. Potential major damage to the dam, abutments and river channel or eventual dam break.	
Scenario	**HYDROLOGICAL**
Graphical Scheme	



More likely factors	Less likely factors
Inadequate freeboard or wave Wall (Parapet).	Dam toe protection.
Hydrology Flood Study not updated.	Large reservoir flood storage and freeboard.
Inadequate design capacity of spillway.	Massive good quality rock along the dam toe.
Gated spillway with inadequate gate operation rules.	Provision of emergency uncontrolled spillway.

Actions for Improving the Hydraulic Safety
Update Hydrology Flood Study.
Reevaluate Inflow Flood Accommodation criteria.
Evaluation of existing conditions at the dam toe.
Evaluation the reservoir operation rules during flood season. Including all outlets facilities.
Evaluate the increase of flow capacity by using additional/non-conventional spillways, auxiliary or emergency spillways.
Provision of flood forecasting system in the river basin.

More information
More information on Spillway Capacity, see Chapter 3, Volume 1.

Failure Mode 2	EMBANKMENT DAM OVERTOPPING DUE TO INSUFFICIENT SPILLWAY CAPACITY

Description

The spillway has insufficient hydraulic capacity to pass the design flood and maintain a safety free-board. This can result in overtopping of the dam´s crest. Flow over the crest washes out material in the downstream slope of the embankment and causes massive erosion that progresses leading to slope instability, breach formation and dam failure.

Scenario	HYDROLOGICAL

Graphical Scheme

More likely factors	Less likely factors
Inadequate freeboard.	Existing protection measures for overtopping in the downstream face.
Hydrology Flood Study not updated.	Large reservoir flood storage.
Inadequate design capacity of spillway.	Provision of emergency uncontrolled spillway.
Reservoir Flood control volume reduced.	
High erosion potential of downstream slope of dam.	

Actions for Improving the Hydraulic Safety
Update Hydrology Flood study.
Evaluate increase of the spillway capacity by using additional/non-conventional spillway.
Evaluation of existing conditions at the downstream face and dam toe.
Intensify protection measures along the downstream face and dam toe.
Adequate a reach of the embankment as a controlled overtopping section.
Establish a system of flood forecast in the river basin.
Evaluation of the reservoir operation rules during flood season including all outlets facilities.

More information
More information on Spillway Capacity, see Chapter 3, Volume 1.
Failure Mode 1 recreated in Case Study B3, Taum Sauk Dam, Volume 2.

Failure Mode 3	DAM OVERTOPPING CAUSED BY GATE FAILURE

Description

In the event of an extreme flood or even normal flow conditions, the spillway gate does not operate because of mechanical or electrical failure. That can result in increased reservoir level and eventual dam overtopping, deep erosion of Embankment dam on its downstream slope or toe of concrte/masonry dams and eventually, the dam failure.

Scenario	HYDROLOGICAL - NORMAL

Graphical Scheme

More likely factors	Less likely factors
Non-trained operators /absence of standard operation manuals.	Operation is well defined. Proved rules for gate operation exists. Trained personnel.
Operation gate failure (Electromechanical).	Existence of surveillance and maintenance program.
No routinely inspection, or maintenance or testing of gates.	Redundancy in number of gates.
Deterioration and ageing of gates.	Downstream face and toe protection.
Poor maintenance of gates	Provision of parapet.
Power supply is non-dependable. No backup supply.	Provision of emergency uncontrolled spillway.
Occurrence of flash floods in the River basin	
Limited freeboard / Small reservoir.	

Actions for Improving the Hydraulic Safety
Update Hydrology Flood study.
Detailed evaluation of the electromechanical equipment / Include testing.
Installation of a backup power supply.
Training plans for the operation personal.
Evaluation of freeboard under limited spillway discharge.
Provide / update dam operation manual.

More information
For Dam Overtopping, see Chapter 2 and 3, Volume 1.
For Gate Failure see, Chapter 3, Volume 1 and Appendix F, Volume 2.

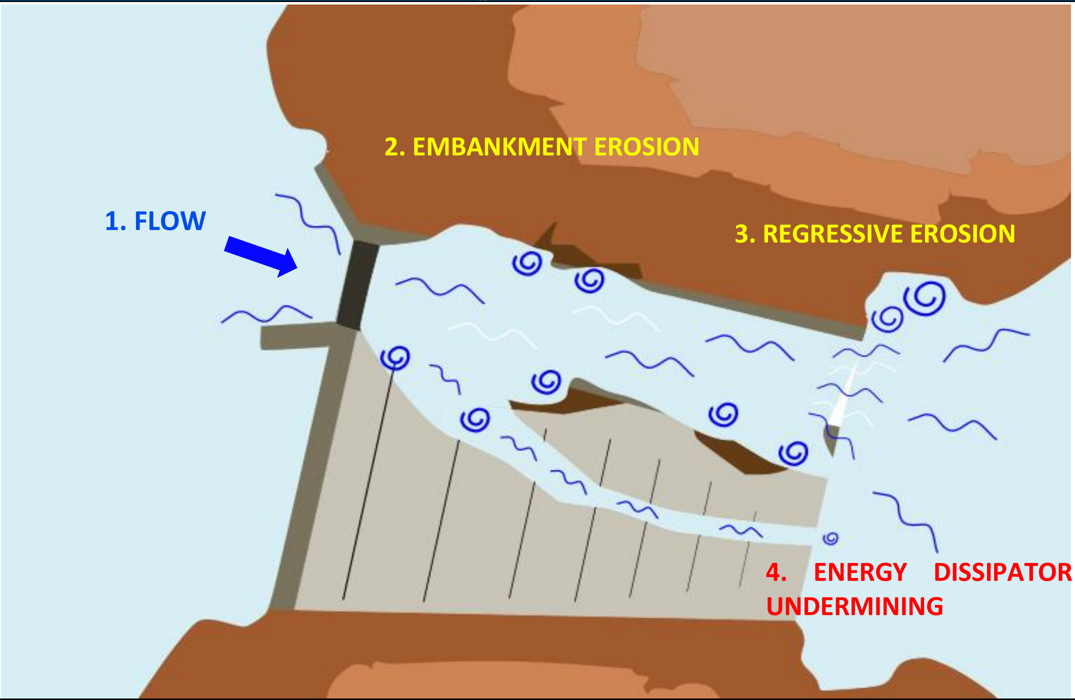
Failure Mode 4	SPILLWAY BLOCKAGE BY DEBRIS ,GATES JAMMING AND DAM OVERTOPPING
Description	
During a flood, a significant load of floating debris gets into the spillway approach channel, causing a gate jamming, gates do not open, spillway is non operative and reservoir level rises leading to dam overtopping. Flow over the downstream face washes out material or eroded toe of concrete/masonry dam and causes massive erosion that progresses upstream. Water cuts the crest of the embankment eroded toe of concrete/masonry dam until the dam breaks.	
Scenario	HYDROLOGICAL
Graphical Scheme	

More likely factors	Less likely factors
Rapid increase in level of water in reservoir.	Provision of emergency uncontrolled spillway.
High production of debris/ice in river basin.	Existence of floating barriers in the reservoir (Tuff Booms).
Inadequate design/location of spillway.	Downstream face and toe dam protection.
Limited freeboard / Small reservoir.	Existence of dam parapet.

Actions for Improving the Hydraulic Safety
Conservation measures for the river basin.
Evaluate possibility of adopting an emergency spillway.
Develop / update reservoir operation rules.
Evaluate design against debris blockage.
Provision of debris dam upstream of the reservoir / dam.
Provision of floating barriers in the reservoir
Program of Supervision/Maintenance in the reservoir and river basin to control debris in the reservoir.

More information
For Spillway Blockage, see Chapters 2 and 3, Volume 1.
For Gate Jamming, see Chapters 2 and 3, Volume 1 and Appendix F, Volume 2.
For debris control, see Appendix C, Volume 2.

Failure Mode 5	CHUTE WALL OVERTOPPING
Description	
In the event of a flood larger than design flood, flow from a spillway can overtop the spillway/chute training walls and can initiate a process of erosion of the back fill supporting the concrete structure. An intense erosion process can affect the foundation of the spillway concrete slab and the contact with the embankment. Eventually the spillway concrete chute collapses. Regressive erosion leads to dam's break.	
Scenario	HYDROLOGICAL
Graphical Scheme	



More likely factors	Less likely factors
Spillway layout - close to Embankment/Abutments with overburden material or adverse geological features.	Apparent good historic performance of the spillway.
Hydrology Flood Study not updated.	Good planning of overall maintenance.
Insufficient freeboard of chute walls.	Concrete works apparently in good conditions
Experience of Operation of the spillway only for low flows.	Competent spillway rock foundation.
Inadequate structural/hydraulic design.	Adequate operation of bottom outlet.
Hydrological characteristics of river basin induce large floods with rapid response.	Design based on physical hydraulic model investigation / Modern design...
Unavailability of flood forecast in the upper reaches.	Spillway seldom operates.
Bottom outlet with limited capacity.	
Aging and poor maintenance of bottom outlet gate.	

Actions for Improving the Hydraulic Safety
Update Hydrology Flood study.
Carry on an evaluation of hydraulic safety of the dam-reservoir system: spillway and bottom outlet.
Check spillway's operation with hydraulic models (numerical or physical) under different hydraulic heads.
Consider use of control gates.
Provision of flood's forecasting in the river basin.

More information
For Chute walls overtopping, see Chapter 3, Volume 1.
Failure Mode 5 recreated in Case Study B4, El Guapo Dam, and Volume 2.
For design of Spillways using Physical Models, see Appendix D, Volume 2.

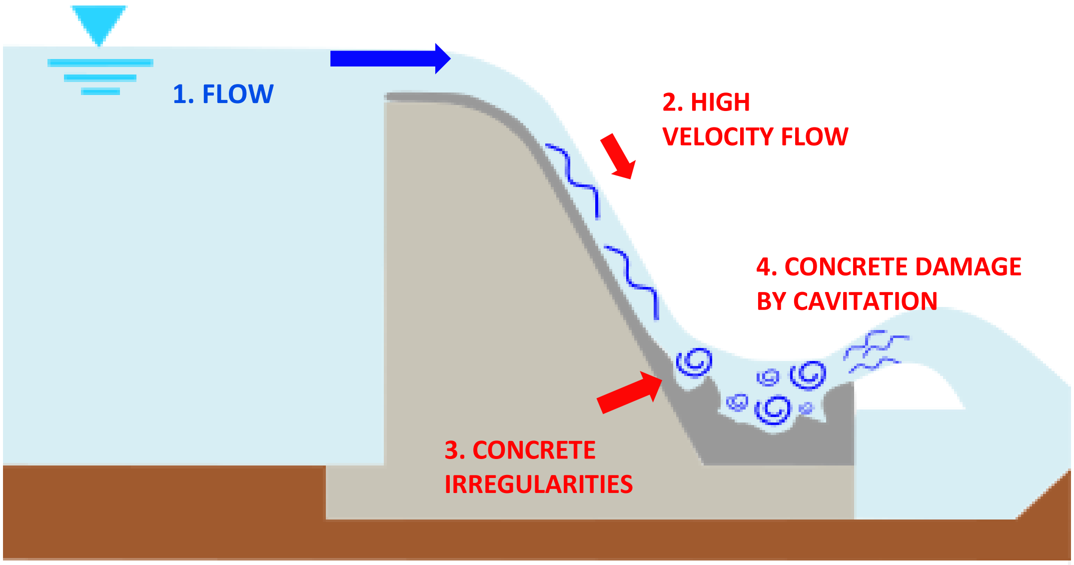
Failure Mode 6	CAVITATION DAMAGE IN CHUTE/GLACIS AND ENERGY DISSIPATORS

Description

Extreme flood or even operation with normal flow, either uncontrolled or gated spillways.
As a consequence of the occurrence of a cavitation process on the spillway glacis/chute, the progressive loss of material develops, which may affect the spillway discharging capacity, the loss of availability and, eventually, the fracture and unavailability of the spillway.

Scenario	HYDROLOGICAL AND NORMAL

Graphical Scheme



More likely factors	Less likely factors
Presence of irregularities on the concrete surface.	Low frequency spillway operation / reduce flows.
Non uniform geometry or non-prismatic sections.	Smooth surface and hydrodynamic design.
High velocity flow in chute/glacis.	Hydraulic design based on physical models.
No aeration devices present.	
Difficult access to inspect the spillway.	
Spillway built before 60's when knowledge of cavitation processes was low	

Actions for Improving the Hydraulic Safety
Carry on evaluation of hydraulic safety of the dam-reservoir system: spillway and outlets for large floods.
Evaluate the design and incorporation of aeration devices / physical modeling investigation.
Execute detailed field inspections of the spillway and its energy dissipation arrangements.
Make concrete repairs as detected (short time of response), to avoid rapid increase of damage.
Consider having an additional spillway whether operation is very frequent.
Improve access of personnel for inspections and repairs.

More information
For Cavitation Damage in Outlets and Spillways, see Chapters 3 and 4, Volume1.
For Cavitation Damage in Energy Dissipator, see Chapter 5.
For Cavitation damage and cavitation control. See Appendix B1, Guri Spillway, Volume 2.
For design of Spillways using Physical Models, see Appendix D, Volume 2.

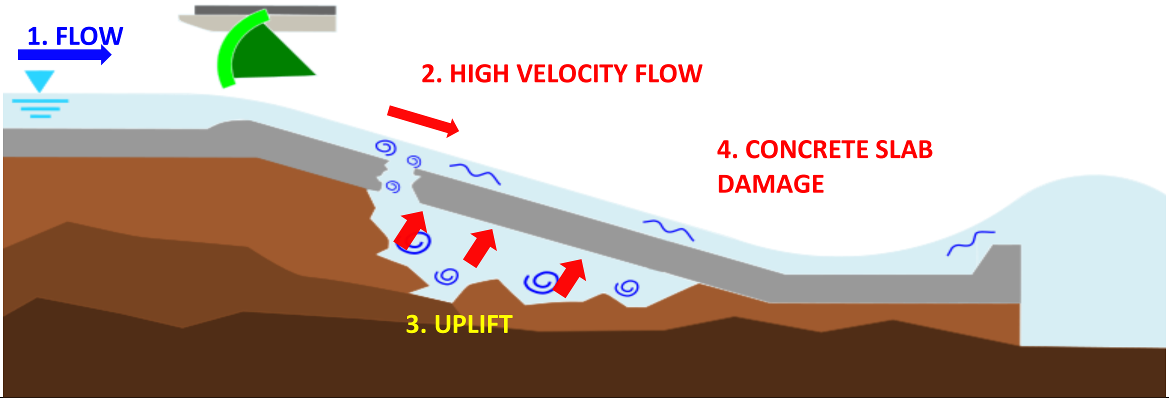
Failure Mode 7	HYDRODYNAMIC PRESSURE (HYDRAULIC JACKING) IN CHUTE AND ENERGY DISSIPATORS

Description

Frequent or extreme flood or even operation with normal flow. Either uncontrolled or gated spillways. Process due to high velocity flow on an irregular concrete surface. An offset in the slab (joint or crack) induces a point stagnation pressure. According to state of opening in the slab, water can pass to foundation material with high pressure, and significant uplift acts on the slab so larger displacement occurs. Foundation is eroded according to erodibility of material, piping occurs. Suddenly the slab collapses and erosion intensifies as gully formation and head cutting. Potential loss of conveyance capacity, spillway loses its function.

Scenario	HYDROLOGICAL AND NORMAL

Graphical Scheme



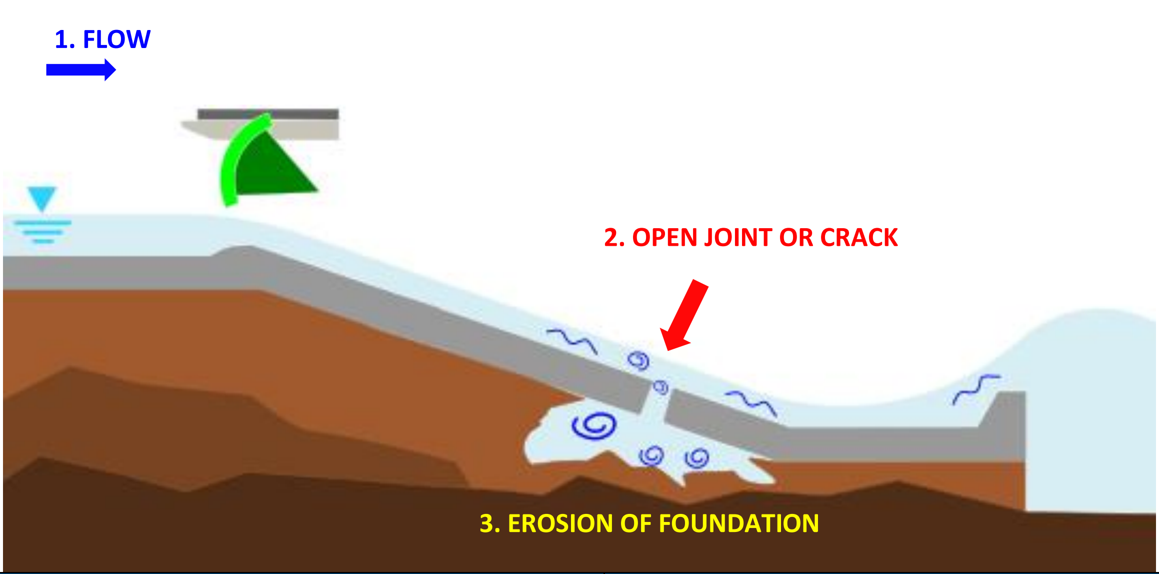
More likely factors	Less likely factors
Open joints or cracks in concrete slab.	Adequate design of slab.
Lack of waterstops in joints.	Existence of a surveillance and maintenance program.
Lack of inspection during construction.	Detailed provision of joints and seals.
Displacement of joint due to temperature, uplift, other reasons.	Foundation of good rock quality.
Poor construction.	Drainage system of the concrete slab.
Reduce weight of slab or lack of anchoring to foundation.	
Frequent operation of spillway.	
High velocity flow in chute / High slope of invert.	

Actions for Improving the Hydraulic Safety
Assess flow characteristics in the chute by numerical/physical modeling.
Improve anchors for slab to rock.
Use special materials for concrete slab to improve impermeability and durability.
Improve access of personal for inspections and repairs.
Inspection of the spillway chute slab, foundation and adjacent works, especially after annual flood season.
Design of aeration devices / physical modeling investigations.
Inspection of the spillway chute slab, foundation and adjacent works.

More information

For Hydraulic Jacking in Chutes and Energy Dissipators, see Chapters 3 and 5, Volume 1.
For Hydraulic Modeling, see Appendix D, Vol2

Failure Mode 8	EROSION OF CHUTE FOUNDATION
Description	
Frequent flood or even operation with normal flow. Either uncontrolled or gated spillways. Process due to high velocity flow over a concrete chute surface with open joint or cracks. Water is forced to pass down to foundation material. Uplift acts on the chute slab. Water follows path along contact slab-concrete, through foundation and under drain system. An erosion process is established according to erodibility of material, piping occurs. Progressively the slab loses support and collapses. Erosion intensifies as gully formation and head cutting. Potential loss of conveyance capacity, Spillway loses its function. This disruption of spillway's function affects hydraulic safety of dam-reservoir system.	
Scenario	HYDROLOGICAL AND NORMAL
Graphical Scheme	



More likely factors	Less likely factors
Open joints or cracks in concrete slab.	Detailed provision of joints and seals.
Poor construction: contraction joints non-sealed.	Under drain system with design filter protection.
Lack of water stops in joints.	Periodic inspection.
Lack of inspection during construction.	Foundation on good rock quality.
Frequent operation of spillway...	

Actions for Improving the Hydraulic Safety
Assess flow characteristics in the chute by numerical/physical modeling.
Improve anchors for slab to rock.
Use special materials for concrete slab to improve impermeability and durability.
Improve access of personal for inspections and repairs.
Inspection of the spillway chute slab, foundation and adjacent works, especially after annually flood season.
Partial demolition of slab and filling holes and cavities in foundation with selected material or concrete.

More information
For Erosion on Chute Foundation, see Chapters 3, Volume 1.
For Prototype experience on erosion on Chute Foundation, see Appendix B1, Volume 2.

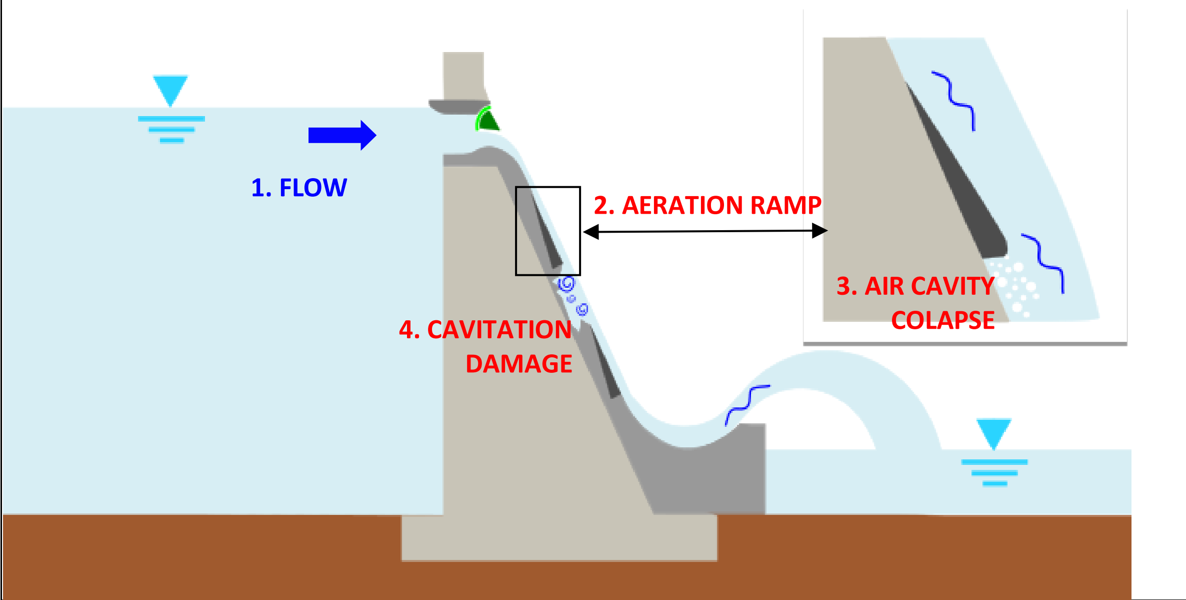
Failure Mode 9	CAVITATION DAMAGE DOWNSTREAM OF AN AERATION DEVICE

Description

Frequent or extreme flood or even operation with normal flow. Either uncontrolled or gated spillways with high hydraulic head. Large depth spillway flows over an artificial aerated chute provided with an aerating device with underestimated dimensions. The aeration ramp is unable of supplying the required air demand and the air cavity collapses, resulting in extensive damage downstream of the aerator due to cavitation. Under this process, chute concrete degradation occurs and progressive loss of material takes place up to fracture/ collapse of concrete element and finally potential loss of spillway's function. This disruption of spillway's function affects hydraulic safety of dam-reservoir system.

Scenario	NORMAL - HYDROLOGICAL

Graphical Scheme



More likely factors	Less likely factors
Model investigations limited by scale.	Reservoir operation at low head over crest.
Wide chute so air is not well distributed widthwise.	Low frequency of spilling.
Small air flow capacity to air duct.	Possibility of repair damaged area.
Ramp geometry: low angle/ low height of ramp.	Use of high-performance concrete or special products to repair damages.
Need for releasing large flows.	
Air passages blocked by debris.	
Poor construction of aerator.	

Actions for Improving the Hydraulic Safety

Assess flow characteristics in the chute by numerical/physical modeling.
Revise design of aeration devices / physical modeling investigation/Improve aerator and air passage design.
Detailed Inspection of the concrete chute surface downstream of the aerator, especially after each flood season.

More information

For Cavitation on Chutes, see Chapters 3, Volume 1.
For Case study on cavitation damage in Chutes, see Appendix B1, Volume 2.

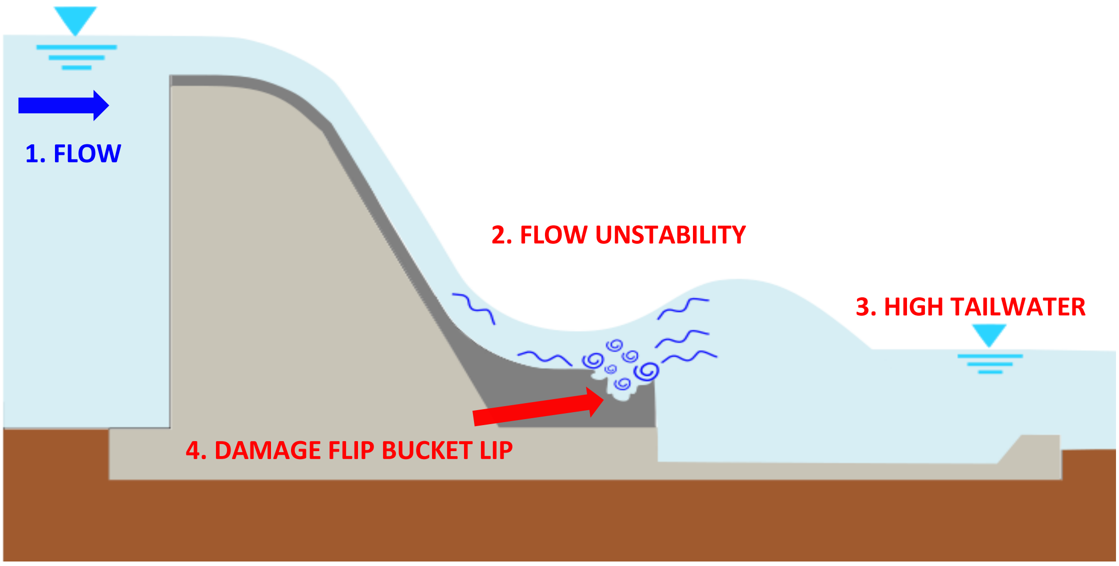
Failure Mode 10	MALFUNCTION OF SPILLWAY FLIP BUCKET DUE TO PARTIAL SUBMERGENCE OF THE JET

Description

Frequent or extreme flood or even operation with normal flow. Either uncontrolled or gated spillways with high hydraulic head. A spillway provided with a flip bucket energy dissipator operating with tailrace water levels higher that the flip bucket lip elevation, creating flow instability capable of generating low pressures, pulsating flow, cavitation and large hydrodynamics loads causing severe damage and costly repair works to the energy dissipator and training walls.
This disruption of spillway's function affects hydraulic safety of dam-reservoir system.

Scenario	NORMAL - HYDROLOGICAL

Graphical Scheme



More likely factors	Less likely factors
Design with flip bucket lip at low elevation.	Flip bucket lip high enough above Tailwater
Bar formation at downstream reaches.	Lowering and/or controlling tail water downstream.
High tail water elevation.	Availability of access to clean materials from bars.
Visual inspection it is not possible due to permanent accumulation of water in the flip bucket.	Reservoir operation to improve flow conditions at flip bucket.
Frequent operation of spillway.	
Large exit angle of flip bucket (40-45º).	

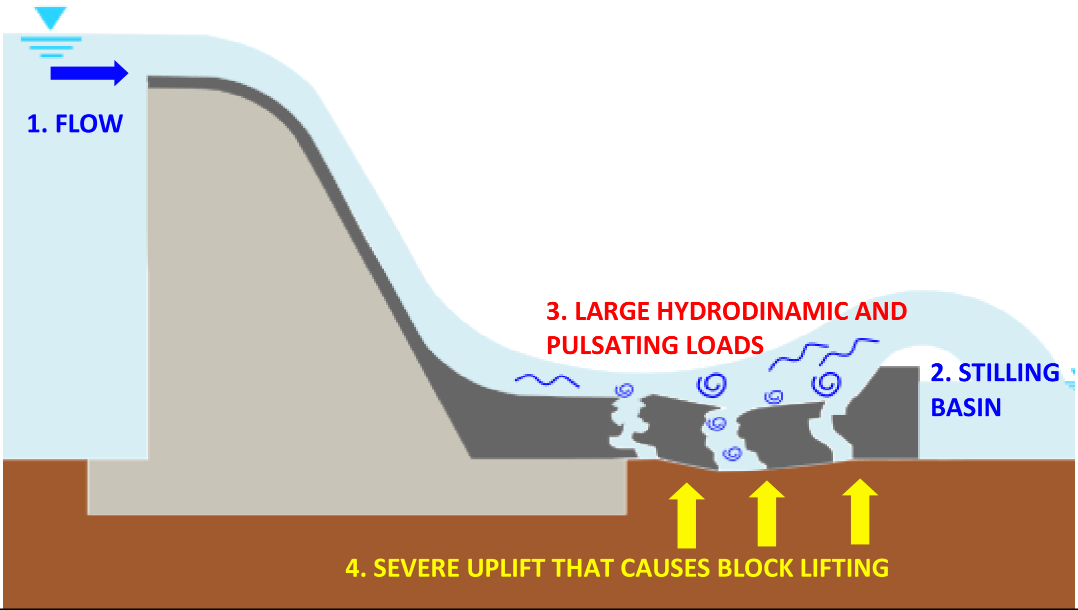
Actions for Improving the Hydraulic Safety

- Evaluation of improvements of tailrace channel.
- Evaluate design of a free jet flip bucket for all operation conditions.
- Evaluate provision of air to the flow by aerating devices.
- Develop a system for repairing concrete damage in dry conditions using bulkheads.

More Information

- For operation of submerged energy dissipation, see Chapter 5, Volume 1.
- For operation of prototype submerged energy dissipation, see Appendix B1, Volume 2.
- For operation of prototype submerged energy dissipation, see Appendix B5, Volume 2.
- For design of Spillways by using Physical and Numerical Models, see Appendix D, Volume 2.

Failure Mode 11	MALFUNCTION OF STILLING BASINS
Description	
Frequent or extreme flood. Either uncontrolled or gated spillways with medium-high hydraulic head. In a hydrologic scenario operation, a large stilling basin present anomalous behavior, noise, explosions associated to large pressure pulsations together with poor construction or design without protection measures at joints against uplift resulting in loss of stability of block slabs, due to large hydrodynamic forces that generate a major failure with very costly remedial measures.	
Scenario	NORMAL - HYDROLOGYCAL
Graphical Scheme	

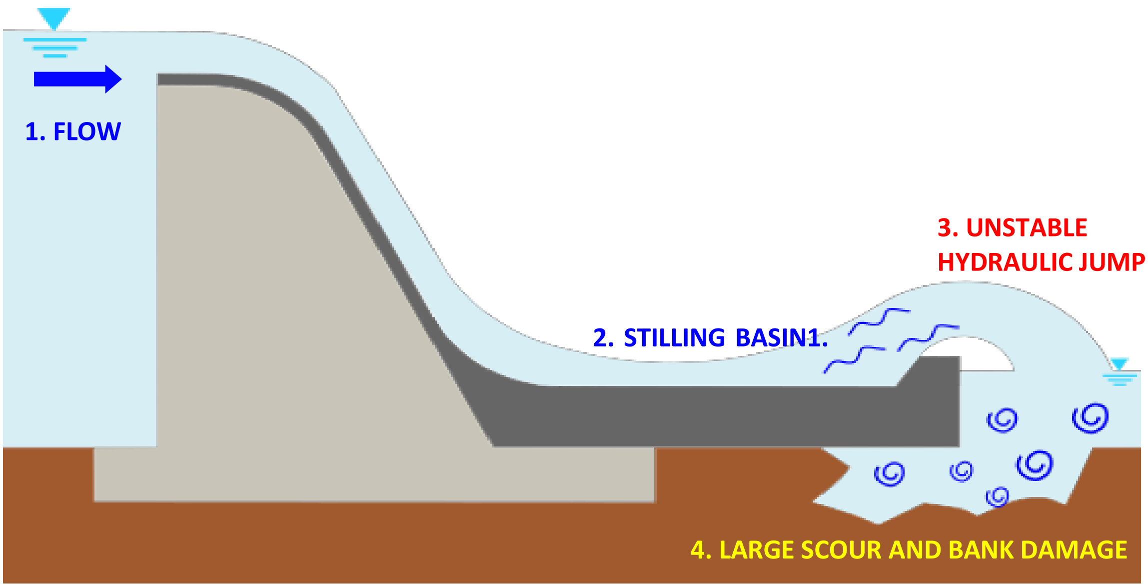


More likely factors	Less likely factors
Unstable Tail water.	Drainage system working well.
Presence of cracks in the concrete slab.	Bulb seals (water stops) in good conditions.
Process of macro turbulence and pulsating flow not investigated.	Diver inspections frequent, after each flood season.
Poor maintenance and inspection.	
System of slab drainage deficient	
Relatively weak geology supporting the slab.	
Weight of slab concrete elements underestimated.	
Experience of stilling basin only for low flows.	
Prolonged operation due to monsoons.	

Actions for Improving the Hydraulic Safety
Monitoring and sub aquatic inspections after monsoon season.
Revaluate energy dissipation type / Further measures.
Improve drainage system.
Revaluate structural integrity of the concrete slab.
Increased weight or resistance to uplift of stilling basin concrete slab.
Program a Field Inspection in the dry

More Information
For estimating of Dynamic loads in stilling basins, see Chapter 5, Vole 1.
For related Failure Modes, see FM 12, Appendix A, Volume2.

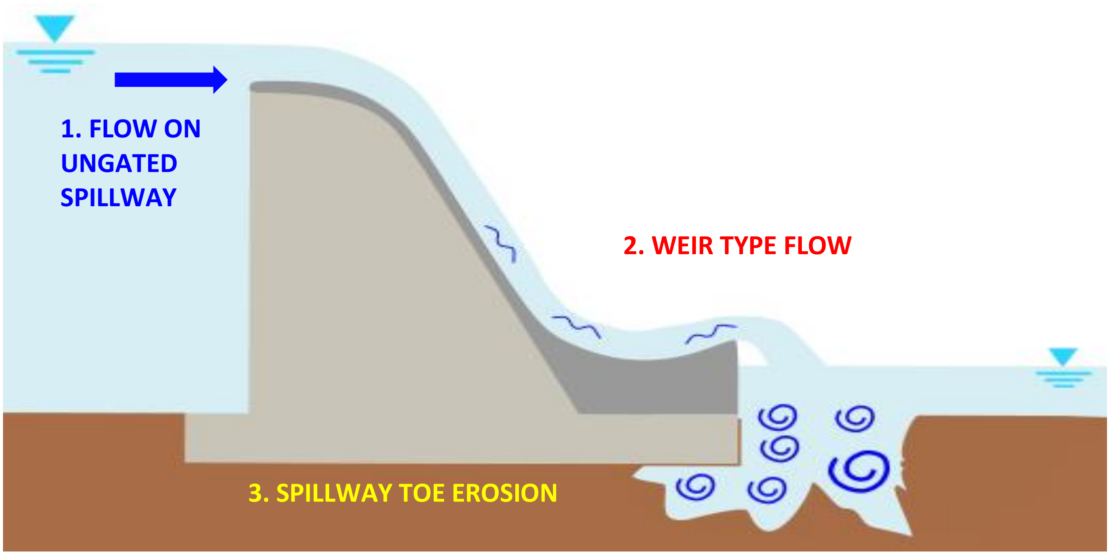
Failure Mode 12	UNSTABLE HYDRAULIC JUMP IN THE STILLING BASIN
Description	
Either uncontrolled or gated spillways with medium-high hydraulic head. In a hydrologic scenario, a severe flood occurs and the hydraulic jump present unstable behavior. Eventually, jump is swept/washed out from the stilling basin generating large erosion to the downstream reaches of the river, including bridges and road infrastructure located nearby.	
Scenario	HYDROLOGICAL
Graphical Scheme	



More likely factors	Less likely factors
Inadequate tail water for a stable hydraulic jump to form for high discharges.	High TW due to the presence of a "run of the river" hydropower plant.
Ongoing scour process downstream, not monitored	Exit channel well monitored and maintained.
Rip rap or banks not sufficient to arrest erosion for the expected flow velocity.	Good bed rock quality at downstream reach.
Resulting high flow velocity and turbulence in the immediate lower river reaches.	
It is a frequent flow deficiency in many projects.	

Actions for Improving the Hydraulic Safety
Assess jump stability and flow characteristics in the stilling basin and tailrace by numerical/physical modeling.
Protect downstream infrastructures, road slopes and bridges.
Build a downstream control weir to guarantee jump stability.
Evaluate all alternatives to control the jump even for design flood
More Information
For related information on hydraulics safety of Stilling Basin, see Chapter 5, Vol 1.

Failure Mode 13	UNGATED SPILLWAY TOE EROSION DUE TO WEIR FLOW AT THE BUCKET LIP
Description	
In a normal scenario a weir- type flow establishes at the flip bucket terminal area, a flow with scour potential initiates a process of erosion at the transition of the concrete structure and the tailrace channel, including concrete structure undermining, unavailability of the structure and costly repair works.	
Scenario	**NORMAL**
Graphical Scheme	

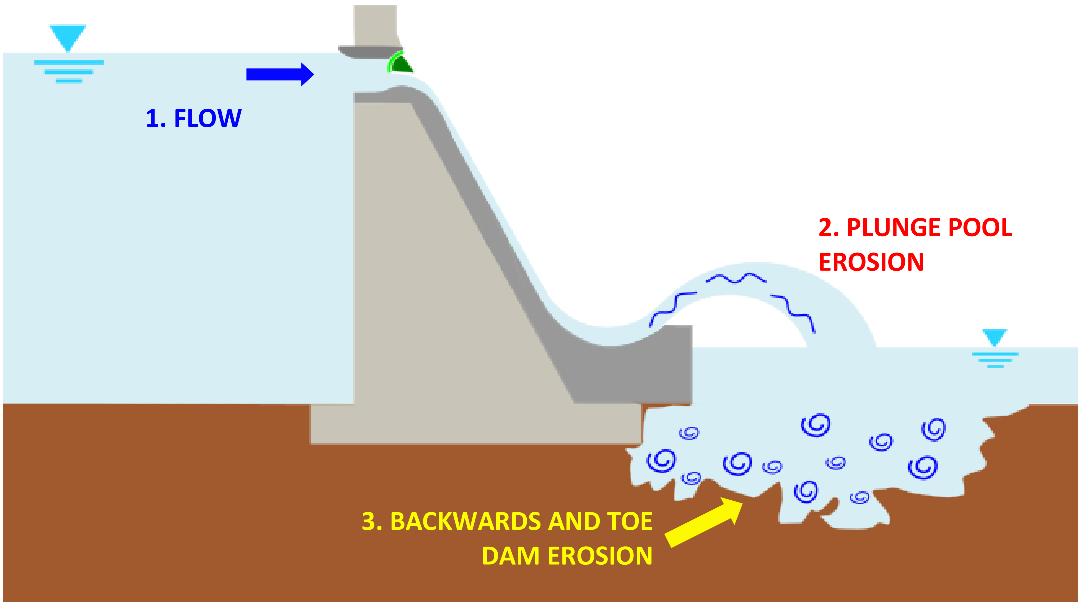


More likely factors	Less likely factors
Frequent spillway operation with low flows.	Existence of key in spillway bucket of adequate depth.
Not so good geology at spillway foundation.	Existence of concrete protection at the spillway toe.
Design is not so robust to protect the flip bucket foundation for weir type flow	
Poor access and difficult conditions for visual inspections.	

Actions for Improving the Hydraulic Safety
Improve access of personal for inspections of the structure.
Program sub aquatic inspections after each large flood
Intensify inspection and monitoring of the prototype pre and post monsoon season.
Evaluation of existing conditions at the spillway toe and the tailrace channel.
Provide adequate protection measures for the spillway toe foundation.

More Information
For Prototype experience on Spillway toe erosion, see Appendix B1, Volume 2.
For erosion at Energy Dissipator, see Chapter 5, Volume 1.

Failure Mode 14	PLUNGE POOL EROSION
Description	
Normal flow lower than design. Controlled or uncontrolled spillways. Excessive use of the spillway, during floods or normal hydrologic scenarios, cause problems as backward erosion and scour in the downstream channel or in the energy dissipation base structure.	
Scenario	**NORMAL**
Graphical Scheme	

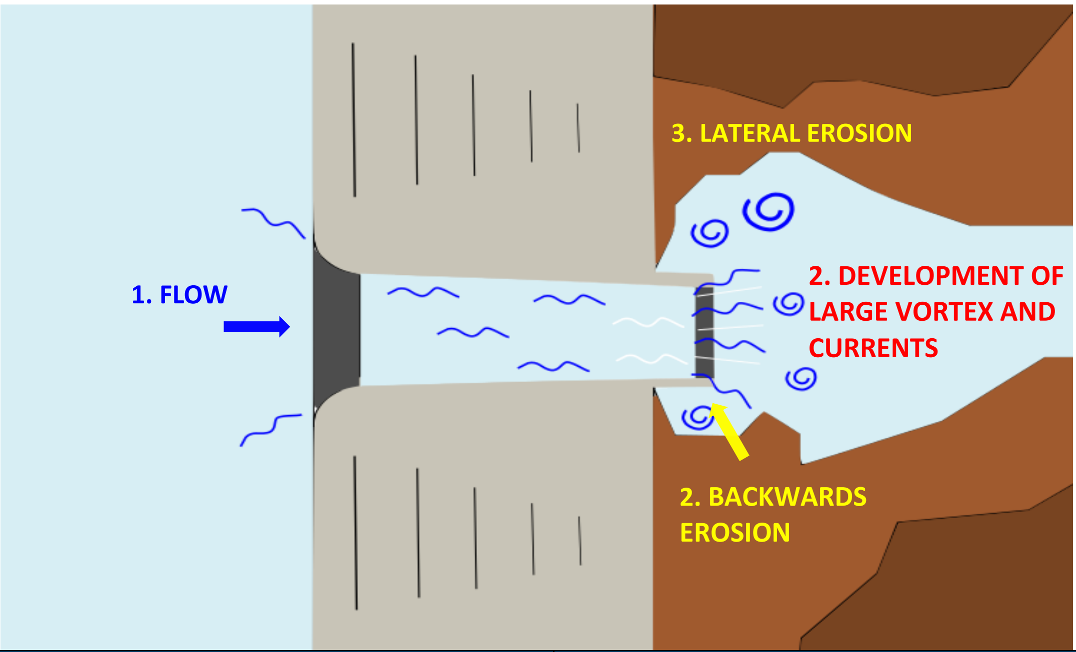


More likely factors	Less likely factors
Incorrect design of energy dissipator.	Adequate protection in toe of spillway.
Overflow capacity exceeded.	Auxiliary or emergency spillway.
Prolonged use of spillway.	Good rock quality at plunge pool.
Difficult access to inspect the spillway and plunge pool.	

Actions for Improving the Hydraulic Safety
Improve access of personal for inspections of the structure.
Intensify inspection and monitoring of the prototype pre and post monsoon season.
Construct an adequate protection for the spillway toe foundation.
Built a control weir type structure in the tailrace to guarantee enough water levels at plunge pool.
Evaluate excavation of a plunge pool to improve energy dissipation

More Information
For spillway erosion at Energy dissipation, see Chapter5, Volume 1.
For Case Studies on spillway toe erosion, see Appendix B1, B2 and B5, Volume 2.
For pre excavated and developing plunge pools, See Appendix B, Volume 2

Failure Mode 15	BANK EROSION DUE TO POOR FLOW EXIT
Description	
In a hydrological scenario, either uncontrolled or gated spillways, a spillway equipped with a flip bucket energy dissipater, create large flow circulation forming at the receiving plunge pool, accompanied by large vortex formation, wave up rush causing large scour on banks, which may affect spillway / dam integrity.	
Scenario	**HYDROLOGICAL**
Graphical Scheme	

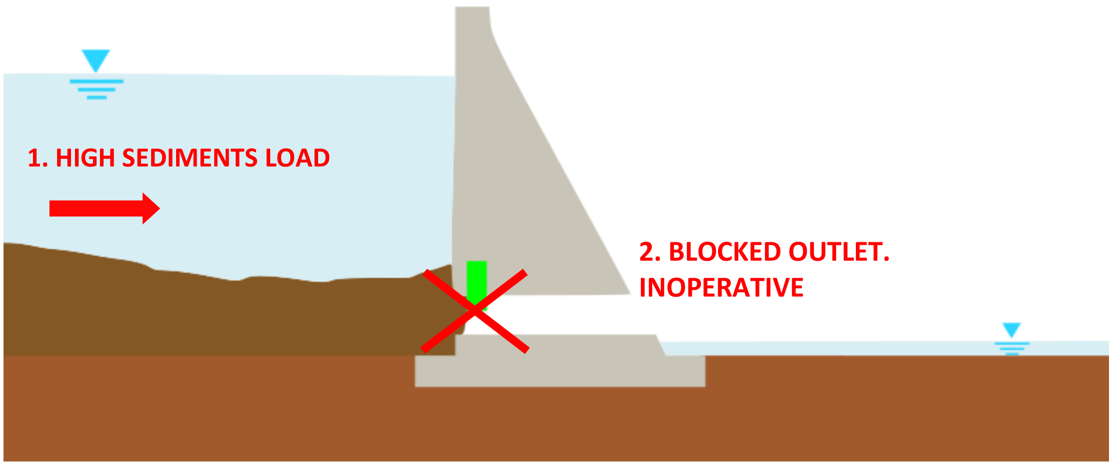


More likely factors	Less likely factors
Large flow spilled/narrow control section in tailrace channel.	Adequate protection of pool, banks and abutments of the main works (powerhouse/ spillway).
Non competent bank material.	Energy dissipated in a pre-excavated plunge pool.
Not detailed investigation in physical models.	Existing access to inspect plunge pool area.
Presence of loose material in the plunge pool.	Low frequency of spillway releases
Asymmetrical flows in the receiving pool.	
Not competent / protected bank material.	
Non symmetrical operation of spillway.	
Presence of bars in the downstream reaches.	

Actions for Improving the Hydraulic Safety
Assess flow characteristics in the plunge pool and tailrace channel by numerical/physical modeling.
Expand narrow control section by suitable excavations including protection works as required.
Monitor plunge pool and bank erosion after the monsoon season.
Improve spillway operation rules (for gated spillways) by doing prototype tests.
Evaluate further protection of banks.
Evaluate lateral slope stabilization works.

More Information
For plunge pool performance see Chapter 5, Volume 1.
For developing and pre-excavated plunge pool see Appendix B1, Volume 2.

Failure Mode 16	BOTTOM OUTLET BLOCKAGE
Description	
Normal flow. As a consequence of the prolonged accumulation of sediments in front of the bottom outlets, they become blocked and inoperative, losing its capacities required for reservoir drawdown, flushing of sediments, environmental flows, and the reduced outflow capacities in the case of extraordinary floods. Partial or temporal loss of outlet operability.	
Scenario	**NORMAL**
Graphical Scheme	

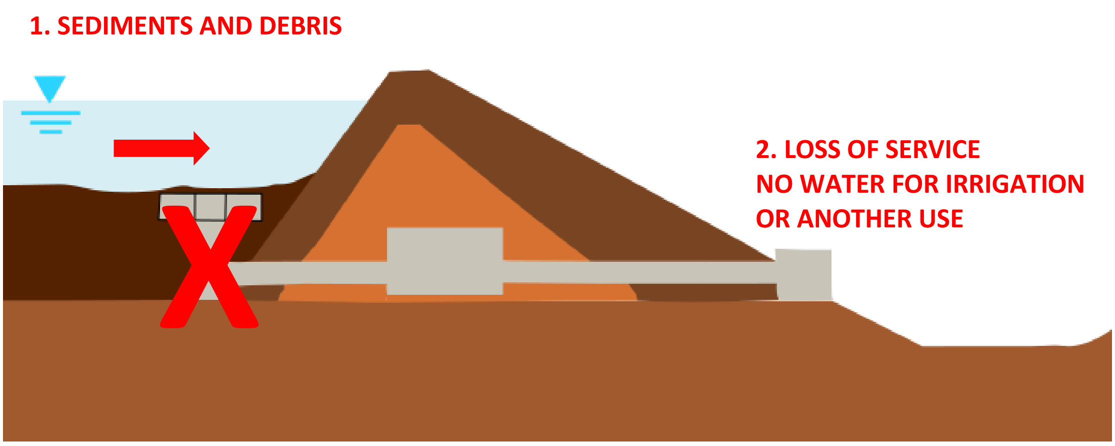


More likely factors	Less likely factors
Uncontrolled use of soils in the contributing river basin.	Good geology of the reservoir rim and river basin.
Presence of high load of sediment.	Upstream dams for sediment trapping.
High trapping efficiency of the reservoir.	Monitoring of forest exploitation.
Inconvenient location of bottom outlet.	Restrictive use of land in river basin
Difficulty in access to its control room.	
Non existing documentation of the outlet design/operative experience.	
Bathymetric monitoring absent	
Fear of operators of using bottom outlet	

Actions for Improving the Hydraulic Safety
Bathymetric survey in the reservoir and sub aquatic inspection nearby the outlet works.
Cleaning outlet intake.
Conservation and environmental measures in the river basin.
Elaborate operation manual of the bottom outlet.
Execution of sub aquatic inspections nearby the bottom outlet.
Improve access to the gates control room.
Maintenance program for all controls and electromechanical equipment.
Monitoring of sediment inflow.
Review performance of bottom outlet and program functional prototype tests annually

More Information
For Blockage of Intakes, see Chapter 2 and 4, Volume 1.
For debris retention by barriers, see Chapter 2, Volume 1 and Appendix C, Volume 2.
For related Failure Mode, see FM # 17, Appendix A, Volume 2.

Failure Mode 17	INTAKE STRUCTURE BLOCKAGE
Description	
Normal flow. Intake blockage as a consequence of accumulation of sediments and debris in front of intake structures resulting from improper reservoir operation, partial or total loss of irrigation intake operability.	
Scenario	**NORMAL**
Graphical Scheme	

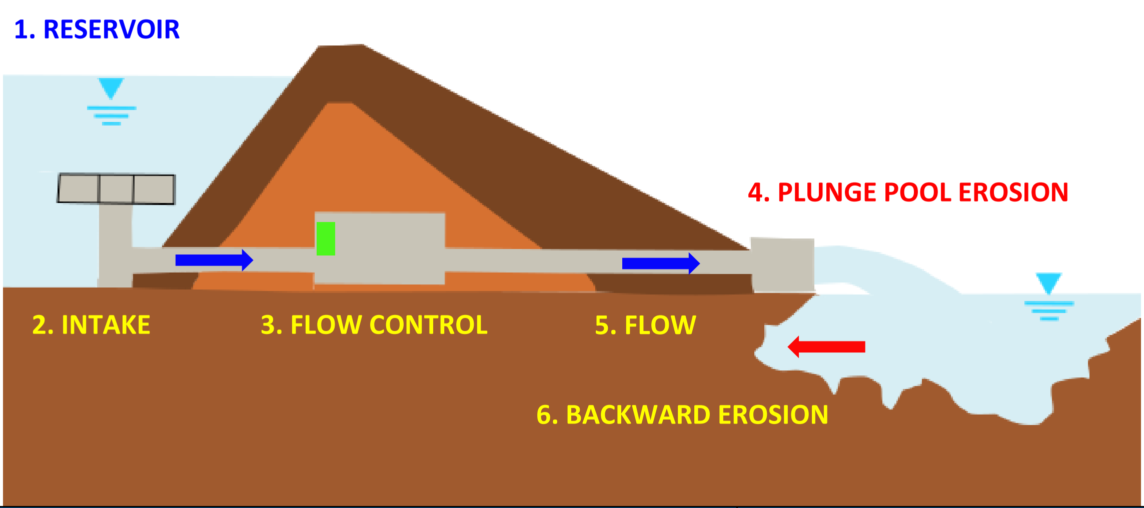


More likely factors	Less likely factors
Poor estimation of sediment loads.	Good geology of the reservoir rim and watershed.
Improper reservoir operation.	Reduced sediment and debris accumulation due to regular operation of the outlet.
Poor hydraulic design of intake structure.	Design of intake for lower velocity of approaching flow.
Non existing historic data of operation.	Operate the reservoir to reduce movement of reservoir sediment to the intake structure.
High sediment trapping efficiency of the reservoir.	Permanent operation of outlet work.
Difficult access for gates control room.	Existence of upstream dams for sediment trapping.
High production of sediment in river basin.	Presence of bottom outlet with adequate routine of operation for flushing sediment.

Actions for Improving the Hydraulic Safety
Bathymetric survey in the reservoir and sub aquatic inspection of intake structure.
Conservation and environmental measures for the river basin.
Cleaning intake from debris and sediment. Evaluate dragging sediments out of reservoir.
Monitoring of sediment inflow.
Maintenance program for all control and electromechanical equipment.
Consider build debris barriers in the reservoir.
Provide / update outlet works operation manual.
Improve access to the gates control room.

More Information
For Blockage of Intakes, see Chapter 2 and 4, Volume 1.
For debris retention by barriers, see Chapter 2, Volume 1 and Appendix C, Volume 2.
For related Failure Modes, see FM# 16, Appendix A, Volume 2.

Failure Mode 18	EROSION AT OUTLET CAUSED BY INADEQUATE ENERGY DISSIPATOR
Description	
Normal flow. Due to the prolonged use of outlet with an inadequate energy dissipater, intense erosion takes place in the downstream channel/canal due to high flow velocity and turbulence, including the dam toe which deepens with time leading eventually to profound damage disabling the operation of the outlet works and possibility of failure of Embankment. Due to the prolonged use of discharge valves and an improper operation of the plunge pool, intense erosion takes place in the tailrace channel and plunge pool due to high flow velocity and turbulence. Erosion can be regressive and deep, leading to outlet structure, causing damage and disabling its operation. Erosion can reach the dam's toe. Partial or total loss of irrigation intake operability.	
Scenario	**NORMAL**
Graphical Scheme	



More likely factors	Less likely factors
Outlet conduit without a properly designed energy dissipater.	Efficient energy dissipation.
Difficult access to inspect energy dissipater / plunge pool.	Robust protection at dam toe.
Outlet conduit operating permanently.	Provision of emergency gate.
Maintenance not practical (24 hour operation).	
Inexistence of a backup intake.	

Actions for Improving the Hydraulic Safety
Inspection / Survey of the downstream area nearby the outlet works.
Evaluation of building a second intake.
Evaluate improvements in energy dissipater at outlet.
Evaluate Improvement of operation rules to reduce scour downstream

More Information
For erosion at Outlets, see Chapter4, Volume 1.
For erosion at Energy dissipators, see Chapter 5, Volume 1.
For Rock Scour quantification by using physical models, see Appendix D, Volume 2.

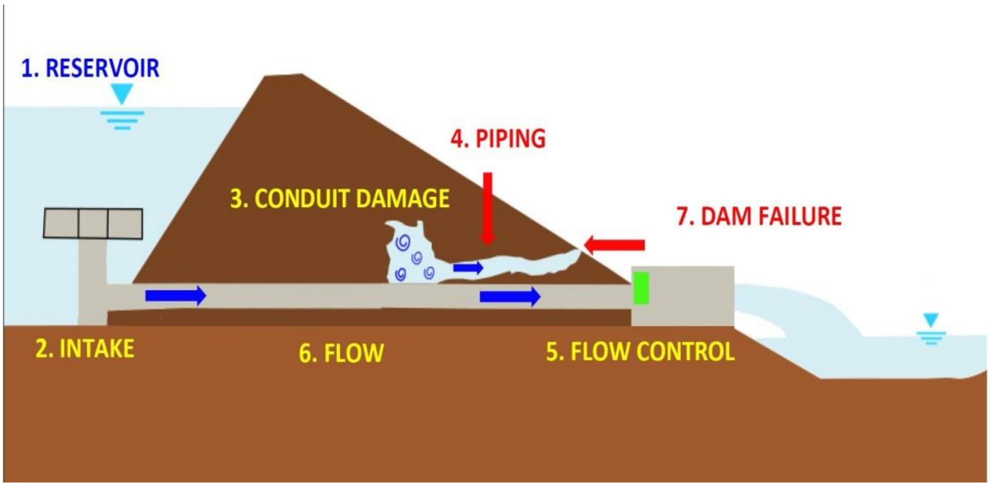
Failure Mode 19	PIPING (OR INTERNAL EROSION) ON THE EMBANKMENT DUE TO LEAKAGE FROM THE PRESSURIZED CONDUIT

Description

During a normal operation scenario, a conduit operating under pressure, suffer from a structural damage, generating a flow from the conduit to the embankment. This flow is capable of generating piping and loss of integrity of the dam which eventually collapses.

Scenario	NORMAL

Graphical Scheme



More likely factors	Less likely factors
Homogeneous dam section.	Provision for internal inspection.
High velocity flow in the conduit.	Dam section with no signs of deficiencies during inspections
Poor construction practices.	Provision of diaphragm filter at d/s end of conduit
Poor geology in the foundation.	
Aged structure.	
No Provision of embankment instrumentation	

Actions for Improving the Hydraulic Safety
Elaborate a programme of monitoring and surveillance of the dam.
Evaluate re-instrumentation of the dam.
Evaluate use of internal revetment of conduit.
Evaluate provision of external barriers or filters against piping.
Evaluate provision of a structure for inspection/ rehabilitation of the conduit upstream of the control section.

More Information

For experiences of internal erosion due to leakage in embankments, see Chapter 4, Volume 1.

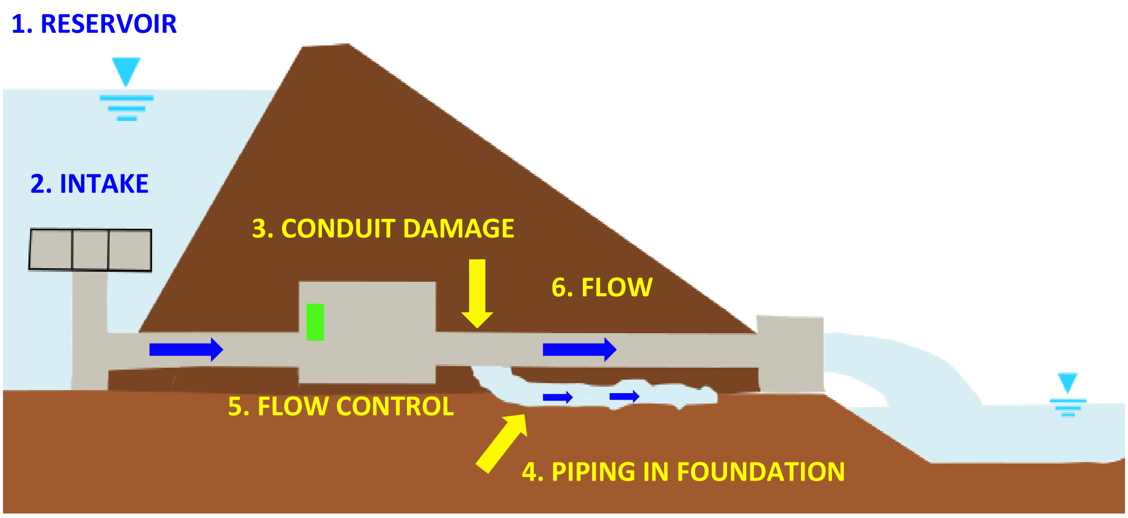
Failure Mode 20	INTERNAL EROSION IN EMBANKMENT DAM DUE TO FLOW INTO A NON-PRESSURIZED CONDUIT

Description

Normal flow operation. Conduit located under the dam with flow control equipment close to intake suffers a structural damage, generating a flow from the conduit to the embankment. Due to the presence of a crack in the concrete conduit, there exists a flow filtration that initiates a process of internal erosion in the conduit foundation. That creates differential settlements and collapse of the conduit, leading eventually to the dam failure.

Scenario	NORMAL

Graphical Scheme

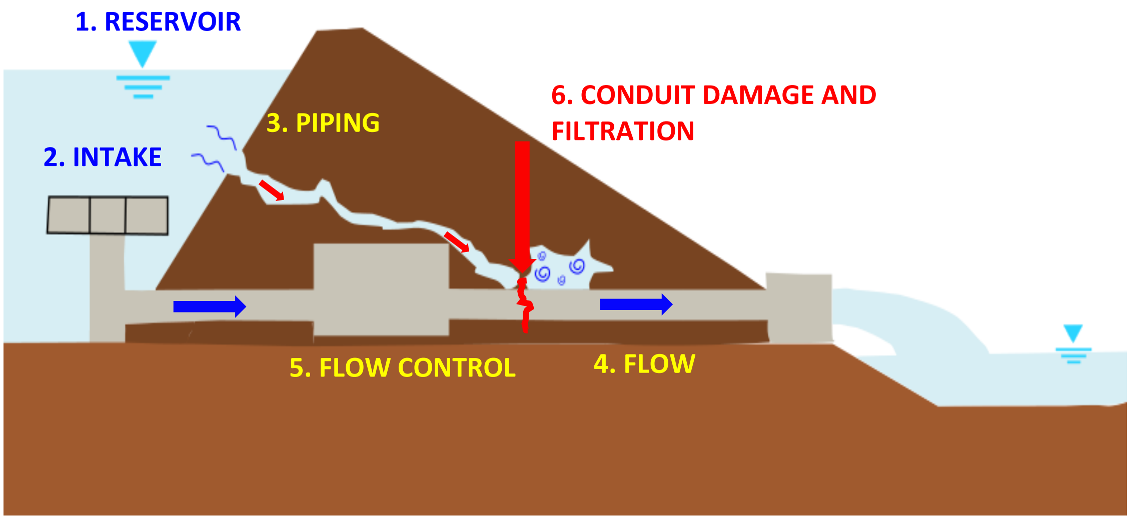


More likely factors	Less likely factors
Poor geology foundation.	Provision for internal inspection.
Poor construction practice: joints, concrete quality.	Dam section instrumented.
There is not an inspection routine established.	Provision of diaphragm filter at d/s end of conduit.
Inadequate selection of the conduit material.	

Actions for Improving the Hydraulic Safety
Evaluation of improving the foundations conditions.
Design of internal improvements of conduit.
Frequent inspection of outlet.

More Information
For experiences of internal erosion due to leakage in embankments, see Chapter 4, Volume 1.
For related Failure Modes, see FM 19 and 21 in Appendix A, Volume 2.

Failure Mode 21	INTERNAL EROSION IN EMBANKMENT DUE TO STRUCTURAL FAILURE OF CONDUIT
Description	
Normal flow operation. Conduit located under the dam with flow control equipment close to intake. Due to the presence of a crack in the conduit downstream of the control section, there is a seepage flow from the dam to the conduit operating with free surface. This seepage will generate migration of materials and piping and will provoque the failure of the upstream slope and eventually of the dam. Partial or total loss of intake and outlet operability...	
Scenario	**NORMAL**
Graphical Scheme	



More likely factors	Less likely factors
Homogeneous earth dam.	Filter within the dam body or around conduit.
Poor construction practices.	Provision of external structural collar.
Difficult access to the outlet.	Provision of diaphragm filter at d/s end of conduit.
Age of dam. Deteriorated condition of conduit material.	Dam section equipped with instruments to detect in advanced the erosion process.

Actions for Improving the Hydraulic Safety
Detailed inspection of discharge conduit.
Evaluate internal rehabilitation of conduit.
Evaluation of structural repairs of conduit.
Design / Provision of instrumentation.

More Information
For experiences of internal erosion due to leakage in embankments, see Chapter 4, Volume 1.
For related Failure Modes, see FM #19, 20 and Appendix A, Volume 2.

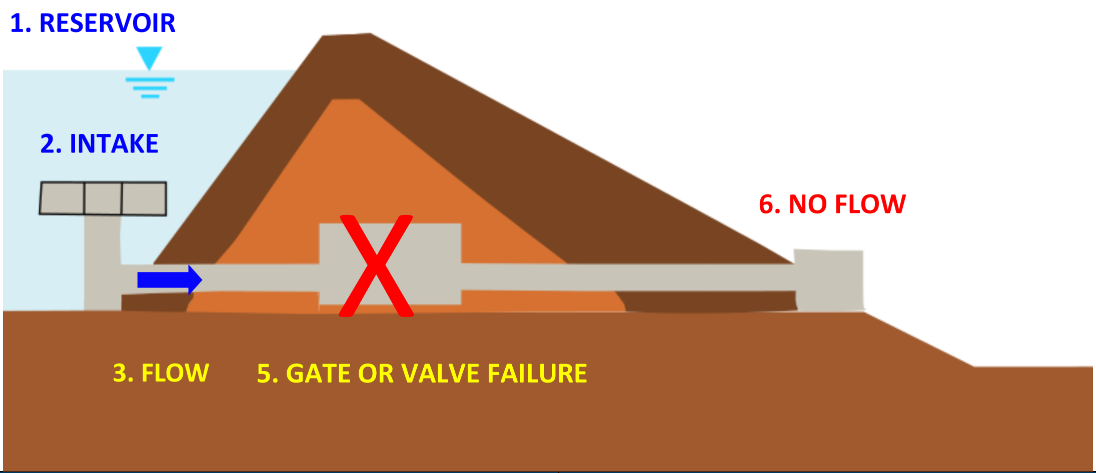
Failure Mode 22	GATES AND/OR VALVES JAMMING

Description

During a normal operation procedure, this failure mode of control structures is associated with the gates, valves, gate operators and control systems. These types of failures do not usually jeopardize the safety of the dam, but limit the ability of the dam to discharge water for normal or emergency releases. This situation produces temporal disruption of outlet's function affecting hydraulic safety of dam-reservoir system.

Scenario	NORMAL

Graphical Scheme



More likely factors	Less likely factors
Non- trained personnel.	Existence of emergency valve and/or by pass.
Inappropriate design.	Preventive maintenance of gate and valve.
Aged structure and equipment	Permanent operation of outlet works.
Poor electromechanical maintenance.	

Actions for Improving the Hydraulic Safety
Provision / update manual of operation of the outlet work.
Improve access to gate control chamber.
Evaluate improvements or modernize the electromechanical equipment.
Incorporation of an auxiliary or emergency gate.
Evaluate a backup system for energy supply to valves or gates.
Evaluate an alternative system for water supply.

More Information
For valves and gates jamming, see Chapter 2, Volume 1.
For valves and gates malfunctioning see, Appendix E, Volume 2.

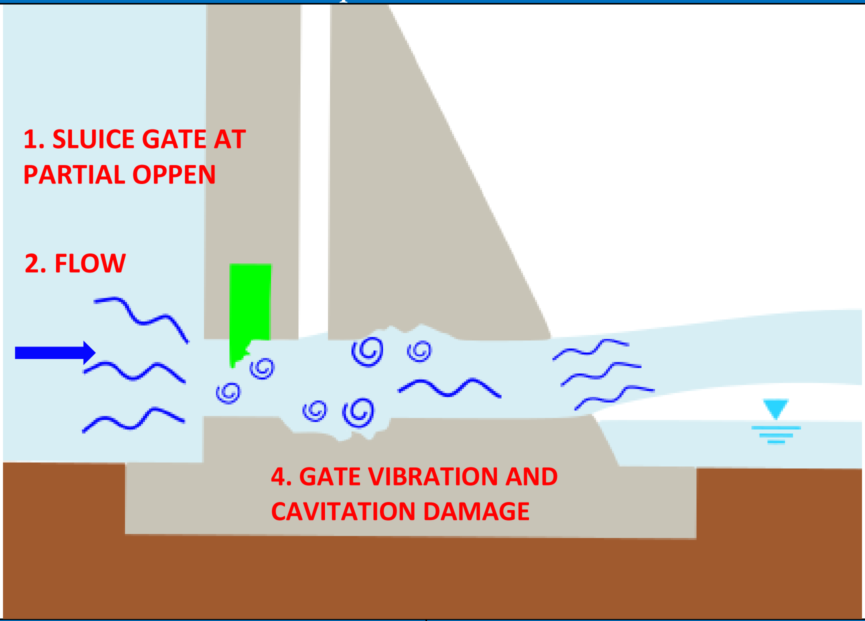
Failure Mode 23	MALFUNCTION OF SLUICE GATE OPERATION AT PARTIAL GATE OPENINGS

Description

In a normal scenario, a sluice outlet gate is opened partially and the flow is controlled at partial opening. Because of low dimensions/obstructions of their ventilation pipes, the flow is not sufficiently ventilated. Due to high air demand, which cannot be satisfied by the ventilating pipe, flow becomes unstable inducing sluice gate vibrations and cavitation damage to the outlet.

Scenario	NORMAL

Graphical Scheme



More likely factors	Less likely factors
Sluice rarely used.	Enough surface gates available for spilling.
Lack of maintenance of the sluice. Not used and not tested for many years.	Large experience of the personnel in operating sluice gates.
Ventilation air conducts of small dimension.	Gate design against vibrations.
Not availability of Operation manuals.	Testing of gates done regularly.
Emergency gates not available.	Sluices well maintained.
Stop Logs not available.	Updated flood study.
Personnel not trained. Fear to open the outlet	
Need to operate the sluice because of high reservoir level and large flow inflows in the reservoir.	

Actions for Improving the Hydraulic Safety
Review of documentation of the bottom outlet/ physical model investigations/air demand.
Update Hydrology flood study to review dependability of outlet work as emergency spillway.
Improve access conditions for inspection and operation.
Review possibility of counting with backup system of the gate.
Program of annual tests of the prototype.
Improve air slots for gate ventilation.

More Information
For ventilation requirements of a gate, see Chapter 4, Volume 1.
For gate vibration due to poor ventilation, see Chapter 4, Volume 1.

APPENDIX B – CASE STUDIES

This page has been left blank intentionally.

B1-Hydroelectric Development of Lower Caroni, Venezuela.

Abstract

The Lower Caroni River Cascade Hydroelectric Development, presently with an operative installed capacity of 14,879 MW (Guri, Macagua and Caruachi) and the not yet completed 2,160 MW (Tocoma), initiated operation in 1960. Spillways for these hydropower stations have a discharge capacity of 30,000 m^3/s with flow concentrations from 260 $m^3/s/m$ to 100 $m^3/s/m$ and heads from 30 to 140 m. In this case study, energy dissipaters of the 4 projects are outlined and the experience of design and operation is briefly presented. The energy dissipaters of Guri Spillway will be discussed in more detail, together with the design measures taken for Guri Final Stage Spillway after malfunctioning of the original structures. A summary of the experiences of the 3 projects located downstream of Guri will also be described.

Hydroelectric Development of Lower Caroni: Guri, Caruachi and Macagua Dams (Edelca; Gómez)

Guri Dam

Guri Final Stage Hydroelectric Project located in Southeastern Venezuela on the Caroni River (Figure B.1-1), with 10,700 MW of installed capacity, represents the backbone of the Venezuelan electricity system, which is part of the Lower Caroni Hydroelectric Development together with Macagua (2,968 MW), Caruachi (2,196 MW) and Tocoma (2,160 MW, under construction) Power plants with total associated capacity of over 17,600 MW (Figure B.1-2). The Caroni River with a mean discharge of 4,800 m^3/s is the largest tributary of the Orinoco River, the third largest river in the world with Guri reservoir ($1,110x10^9$ m^3 of volume capacity) acting as the regulating reservoir while Macagua, Caruachi and Macagua are very close, "runoff the river" power plants.

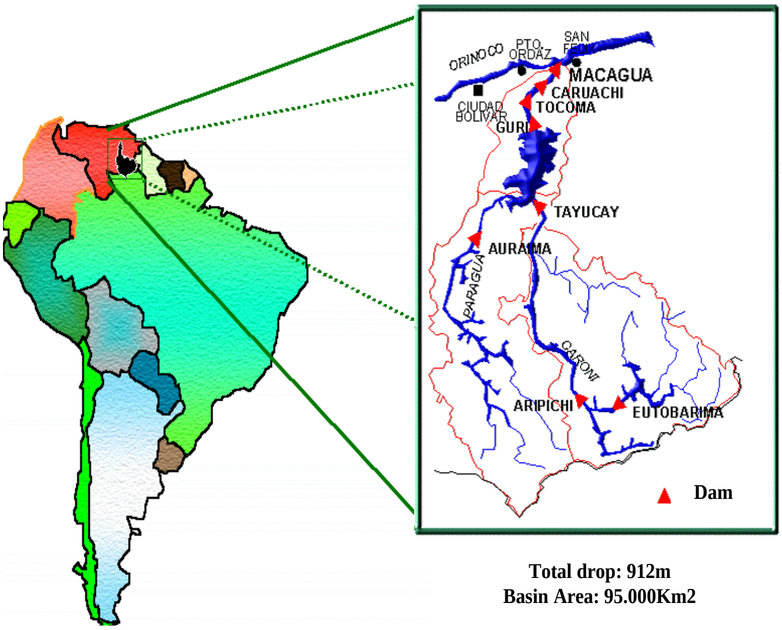
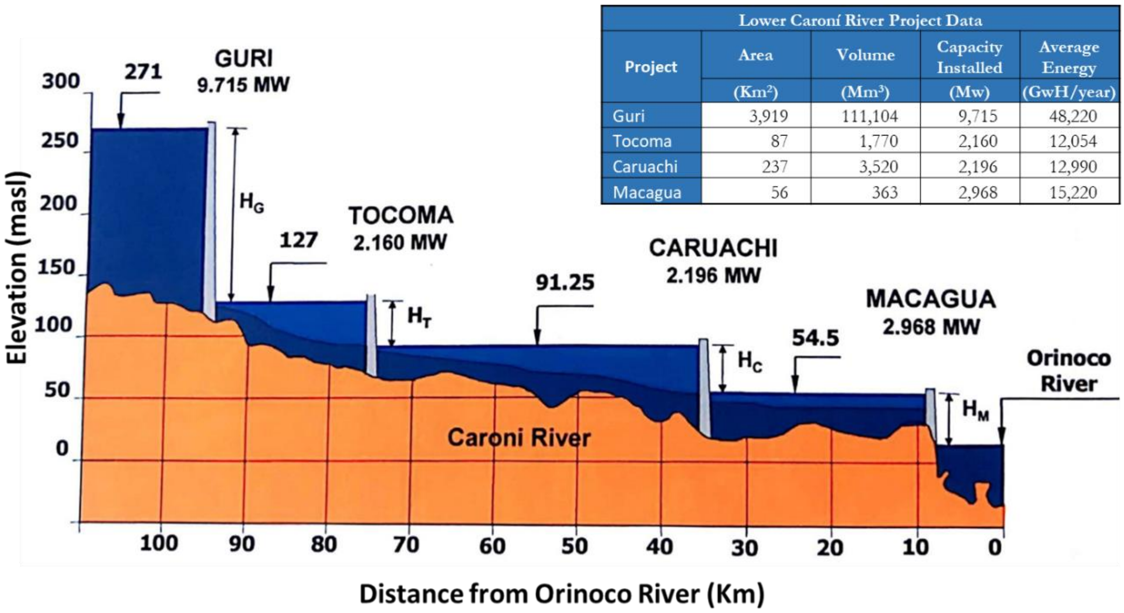


Figure B.1- 1 Location of Caroni River Hydroelectric Development. (EDELCA)



Lower Caroní River Project Data				
Project	Area (Km²)	Volume (Mm³)	Capacity Installed (Mw)	Average Energy (GwH/year)
Guri	3,919	111,104	9,715	48,220
Tocoma	87	1,770	2,160	12,054
Caruachi	237	3,520	2,196	12,990
Macagua	56	363	2,968	15,220

Figure B.1- 2 Lower Caroni River Cascade Hydroelectric Development. (EDELCA)

By now the construction program for the 4 hydro projects has taken a total of 58 years. Guri project was built in two stages. The first stage contemplated construction of Powerhouse No 1, with generation capacity of 2,050 MW by using 10 Francis turbine units of approximately 200 MW each, and a Head of 90 m, a gravity dam, and a spillway with discharge capacity of 29,780 m^3/s, which represents the peak outflow of the PMF design flood. The original spillway consists of three channels, each channel having 3 bays and 3 radial gates, 21m high x 15.24m wide, with spillway crest at EL 195.20, and flip bucket terminal structures with lip elevation at EL 121.22 masl operating under some degree of submergence. Guri spillway is a convergent chute/glacis with flip bucket width of 45m, flipping the jet at some 160 m distance from the spillway end. The spillway was divided into three chutes, each provided with 3 gates for a total of 9 gates and an estimated unit discharge q of 220 $m^3/s/m$. The Final stage of the project consisted in raising all dams and spillways, including radial gates by 55m, equipping the new powerhouse with 10 Francis turbine units of 800 MW each, for a total of 8,000 MW to increase the available power capacity to 10,000 MW.

Guri First Stage

Guri project was built in two stages. Stage 1 was finished in 1978, in which the Caroni River flow was regulated at 40% (see Figure B.1-3 for historical inflow data) with a reservoir area of 800 km^2 to provide water for Powerhouse 1, consisting of 10 Francis Turbines for a total of 2,080 MW. The Peak Inflow of Design Flood for Guri First Stage Project, corresponded to the Probable Maximum Flood, was estimated as 55,200 m^3/s, with a volume of 53.4 x 10^6 m^3 and duration of 23 days. A resulting 40,000 m^3/s was estimated as the routed outflow of the PMF at MWL of 219.5 and used for the design of the spillway.

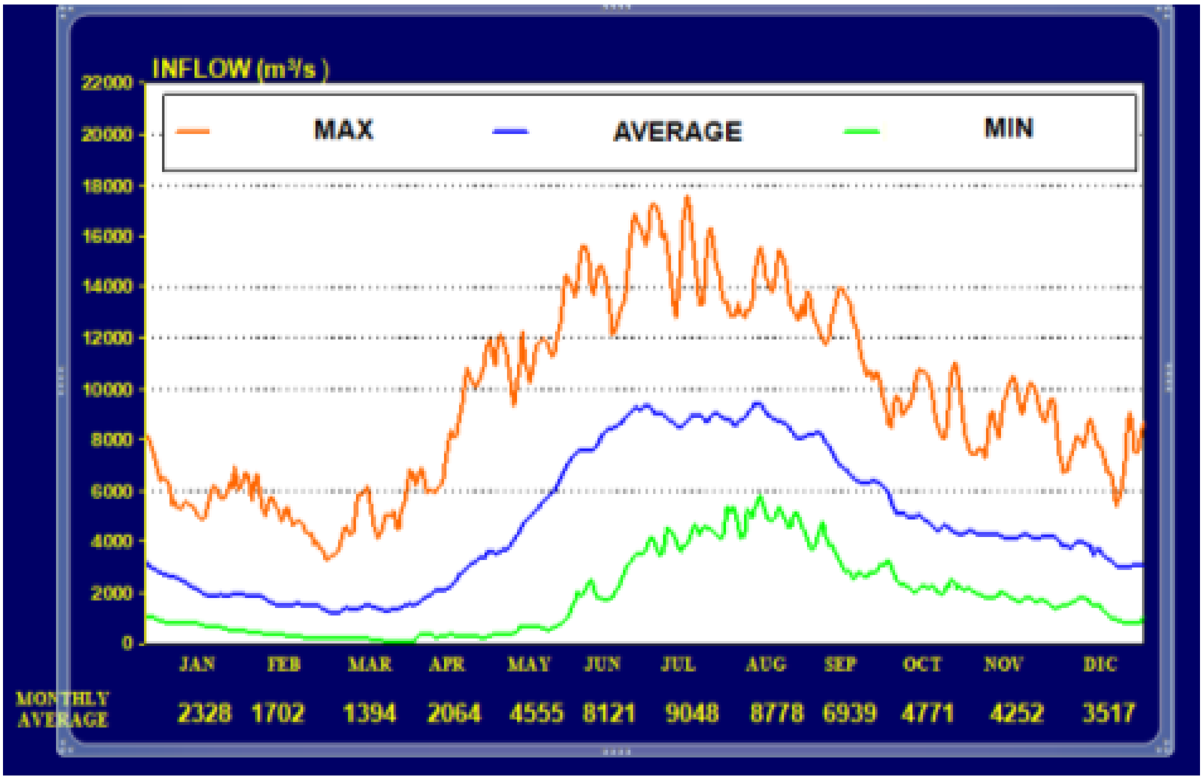


Figure B.1- 3 Historical Inflow Behavior of the Caroni River. (EDELCA)

Guri Project main works

The Spillway was located at the left abutment of the Dam and consisted of 3 channels, each provided with 3 gates 15.24 m x 21.00 m (see Figure B.1-4). Powerhouse 1 selected was next to the spillway, parallel to the spillway base line. Both Powerhouse and Spillway shared the tailrace channel which expands from the powerhouse and reduces in width until merging with Tailrace channel 1, which are 120m wide and 1,100 m long. No pre excavated plunge pool was provided in the project. Left bank area of Necuima Canyon was considered sufficient to receive the water from both the spillway and the Powerhouse. Both flows are conveyed into the Tailrace Channel 1 at original EL of 119 m.

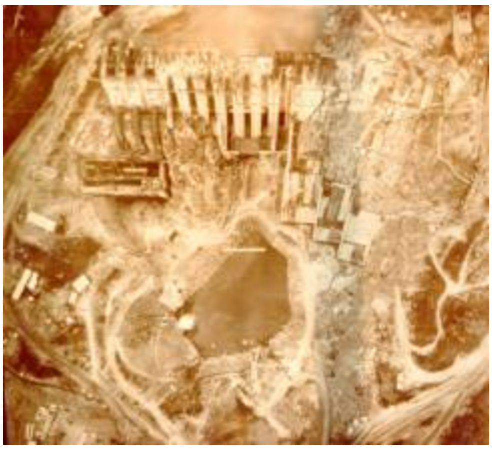



Figure B.1- 4 Left -Construction of the original works, Right - Spillway under construction. (EDELCA)

Design of the original energy dissipaters

Spillway flip buckets were tested in a Physical Model scale 1:250 built at the Hydraulic Laboratory of St Anthony Falls, University of Minnesota. A model at a scale of 1:200 with a rigid bed and a second partial model at a scale of 1:400 were built and tested to solve all the questions posed by the project. As a result of the tests on the erodible physical model bed, the elevation of the bucket lip was selected below the tail water. The analysis of the flip bucket exit angle was done by using the rigid 1:200 scale physical model to reduce the ejector effect. This effect tends to pull the tail water down to water levels that may not guarantee a good performance for the turbines. A lip exit angle of 45 degrees was selected for it provided the maximum trajectory distance of the jet.

Resulting design included:

1. Large lip exit angle of 45°, to obtain maximum jet trajectory.
2. A radius of the energy dissipater of 30 m to guarantee good behavior of the flow to reach the exit area.
3. Flip bucket lips were fixed at El 121.22 for the three chutes.
4. Attention was given to symmetrical operation flow in the spillway rather than in individual chute operation while the remaining two chutes were kept closed.
5. The three spillway chutes of different length, converged into the tailrace channel through a partial depression in the topography, located in front of Chute 1.
6. Top of chute walls at the energy dissipation were designed.

7. Some submergence of the flip buckets lips and walls was accepted with the water elevation for normal powerhouse being at 123.50. When moderate spilling was needed, tailrace water elevations will raise to El 125-126, the flip buckets will operate with some 4-5 m of submergence and water will be over the wall top elevation See Figure B.1-5 and B.1-6).

Figure B.1- 5 Diversion Sluices working together with the 3 Spillways chutes while generation has started in the first units of Power house 1. (EDELCA)

Figure B.1- 6 Walls at flip bucket locations, note that flip buckets lips are submerged (EDELCA)

Operation of the Original Energy Dissipaters

Guri First Stage Spillway operated 5 to 6 months every year, due to the partial regulation (40% of the mean inflow) of the Caroni River. Since the begin ing of its operation, damage was observed in the extreme portion of the 3 flip buckets and walls, necessitating repair works one chute per year as an average. Repair works of the damaged area were done with different materials including epoxy, polymers and others. Finally, high strength Portland cement was selected.

During these tests, negative piezometric pressures up to -5m of water column, were measured accompanied by strong pulsing pressures near the bucket lip, due to flow instabilities created at the exit by the effects of high tail water upon the flow leaving the bucket.

Flip bucket were repaired, some minor geometric modifications in the bucket lip were recommended to improve the behavior of the structure (Figure B.1-7). These measures were adopted in the prototype but did not have the expected results. The Flip buckets operated partially submerged by the tail water. Water levels in the TW were some 4-5- meters higher than the exit lip elevation of the bucket. The photo shows the jet exiting the flip bucket submerged partially by the tail water level (Figure B.1-8 and Figure B.1-9).

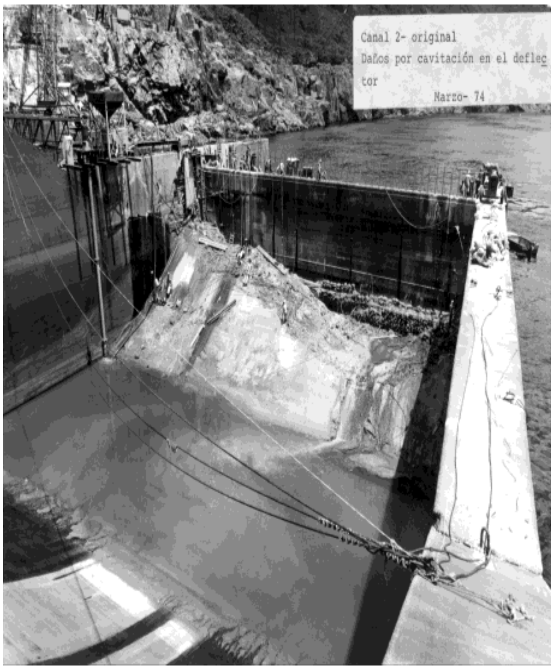


Figure B.1- 7 Repairing works in flip bucket of Chute 2 with the floating bulkhead keeping the flip buckets dry.(EDELCA)

Figure B.1- 8 Once the sluices had been closed and the Powerhouse 1 was in operation, surplus flows were dis-charged by the spillway. (EDELCA)

Figure B.1- 9 Chute 3 damage at Wall 3. (EDELCA)

Figure B.1- 10 Process of placing the floating bulkhead to initiate the repair works. (EDELCA)

Method of repairing works

Damage was diverse but it was possible to repair it with the help of a bulkhead gate provided for inspection of the flip buckets and normally housed at the left bank downstream of the spillway (see Figure B.1-10). The bulkhead gate of 50 m width x 12 m height had been successfully placed with the help of a tugboat to close the open spaces against the frontal face of the flip bucket wall and the lateral walls (see Figure B.1-11 and Figure B.1-12).

In the rainy season of 1978, it was apparent that the jet leaving Chute 2 flip bucket was not flipping and not projecting the jet properly downstream (Figure B.1-13). Bathymetric surveys showed the deep erosion of the flip bucket lip of Chute 2 and the loss of walls 2 and 3. The later will make difficult to place the floating bulkhead downstream (Figures B.1-14).

An underwater survey 6 months later showed some 20m of overhangs extending to the corner of Chute 1 (Figure B.1-15). About 4m of erosion was also identified in Chute 3, at the nearby area of Chute 2. The deep erosion found in the left downstream region of flip bucket of Chute 2, was associated to the presence of an active fault identified in the geology maps of the project (Figure B.1-16).

Model tests carried out in the laboratory demonstrated that during the process in which the jet overcomes the submergence created by the tail water, strong pressure fluctuations were measured at the bucket lip as a result of the high degree of turbulence. These transient conditions in terms of dynamical loads are transferred to the concrete which is unable of absorbing them, hence causing the partial collapse of the structure with time. The impossibility of repairing the submerged flip bucket and the hazard represented by the extent of the erosion indicated the urgent need to work in other options that will raise the flip bucket above the maximum tail water level.

Figure B.1- 11 and Figure B.1- 12 Flow from Chute 2 were not flipping. A Photo taken during a very low tail water elevation, permitted to visualize the lips of buckets 1 and 3, but the bucket lip of Chute 2 totally disappeared as it was removed by the flow. (EDELCA)

Figure B.1- 13 Photo taken where wall 3 is partially removed, and wall 2 is totally removed. In its terminal part, lip of flip bucket of Chute 2 is completely eroded. That situation precluded the placing of the floating bulkhead to initiate repairing works. (EDELCA)

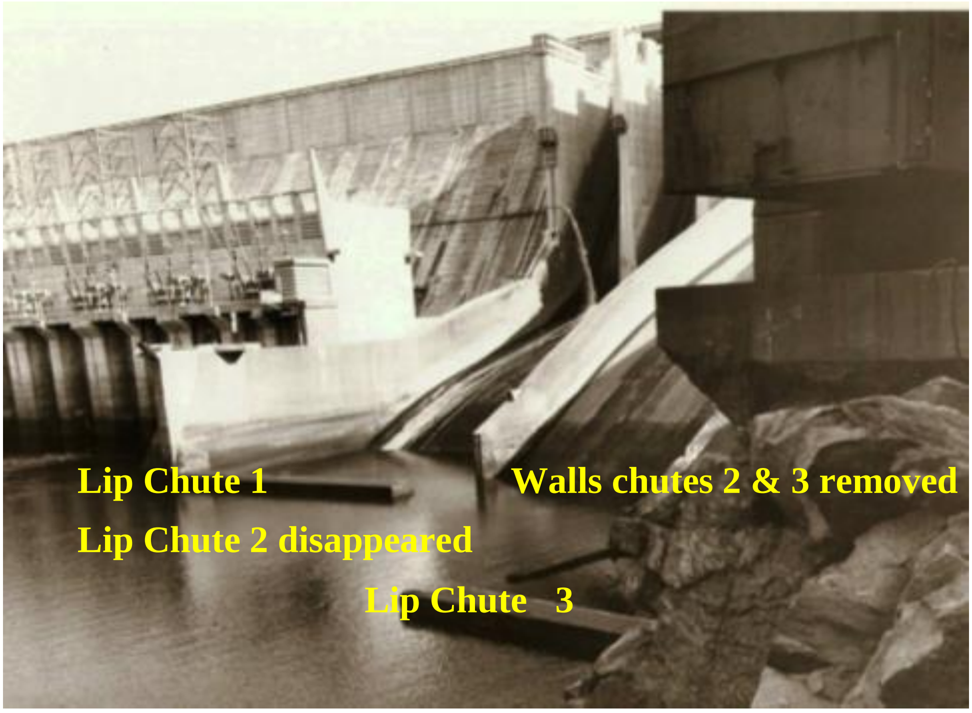


Figure B.1- 14 Photo taken when Tail water levels dropped and the flip bucket lips were exposed. It can be observed walls 3 and 2 were practically removed by the flood; and bucket lip of Chute 2 completely eroded. Repair of Chute 2 was not possible because the walls needed to place the floating bulkhead had been removed by the flood. (EDELCA)

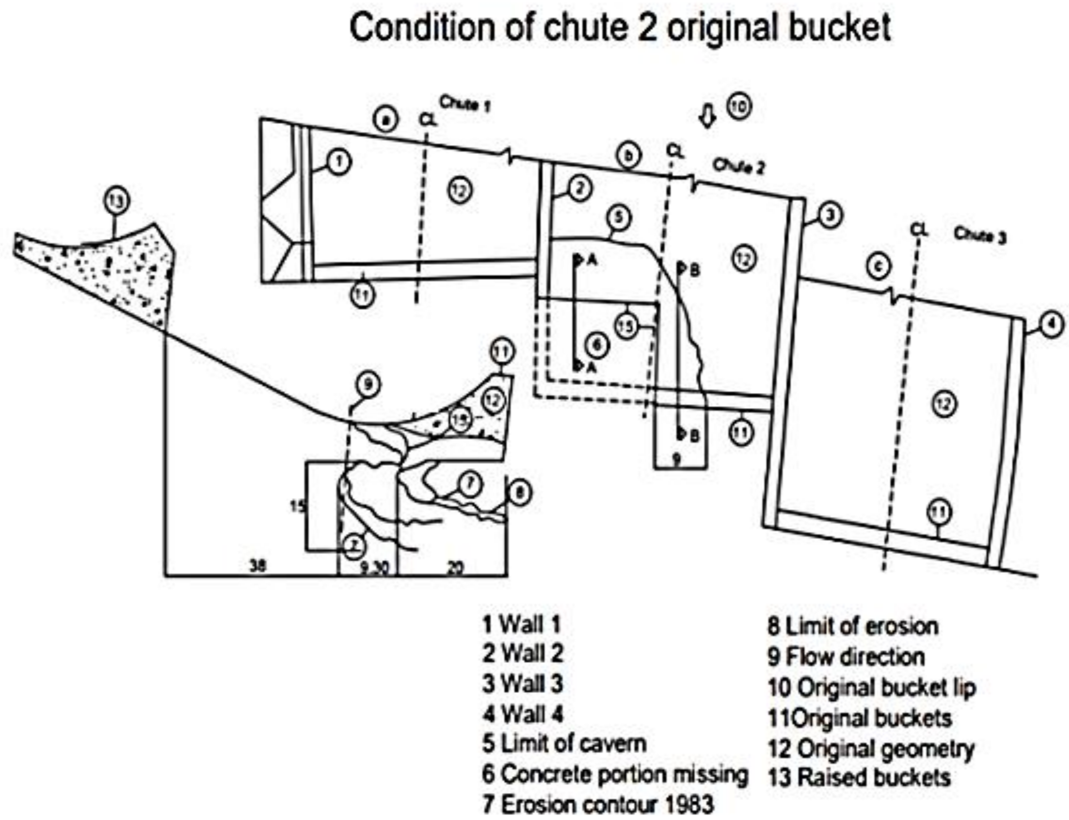


Figure B.1- 15 Diver´s inspection Sketch showing a 20m cavern in the side of the right corner of the flip bucket of the Chute (Marcano, et. al., 1988).

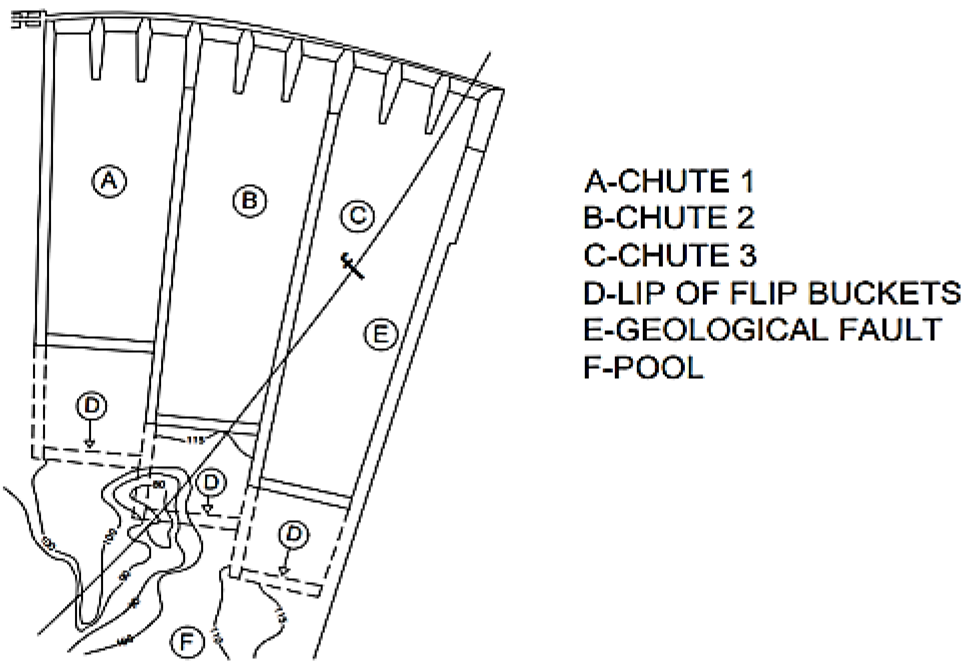


Figure B.1- 16 Bathymetric survey showing inactive fault crossing the right downstream corner of Chute 2 flip bucket (Prusza, 1994).

Causes of the malfunctioning

A second Physical Model was built at St Anthony Falls Hydraulic Laboratory at 1:80 Scales with the purpose of establishing the mechanism of the damage, and devise modifications to the existing design or to the spillway operation which would prevent and alleviate future damages. It was found during the tests that high frequency pressure pulses were measured at the flip bucket lips, where as low as -5m of water column occurred at 13cm upstream of the bucket lip. Hydraulic effects due to submergence were the most responsible for the damage. Submergence occurred in both important areas, first just by the exit lip bucket where flow leaving the lip of the flip bucket, and second this was backed up by tailrace water level thus creating flow instability and develop pulsating forces not able to be absorbed by the concrete. Other severe flow conditions were created by the flow passing over the walls in the adjacent chute to the one under operation, resulting in unbalanced forces on the walls and severely sheared flows capable of developing cavitation phenomena and associated damage (Figure B.1-17). The damage was correlated with this low pressure and a small wedge was tested and proved in the model to raise pressures to positive values (Figure B.1-19). Further tests carried out in EDELCA´s Hydraulic Laboratory demonstrated that during the process in which the jet overcomes the submergence created by higher tail water level, strong pressure fluctuations were measured as a result of the high degree of turbulence and associated sheared flows localized at the bucket lip. This transient condition in terms of dynamical forces are transferred to the concrete which is unable of absorbing them, causing with time fatigue and collapse of the structure. As the damage was so severe, the Chute 2 discharge was temporarily limited to only 2,000 m^3/s, while later these severe damages in both Chutes 2 and 3 were the main reasons to raise the flip buckets for the Guri Final Stage Project. The impossibility of repairing the flip bucket and the hazard represented by the extension of the erosion indicated the urgent need for taking decisions of raising the flip buckets.

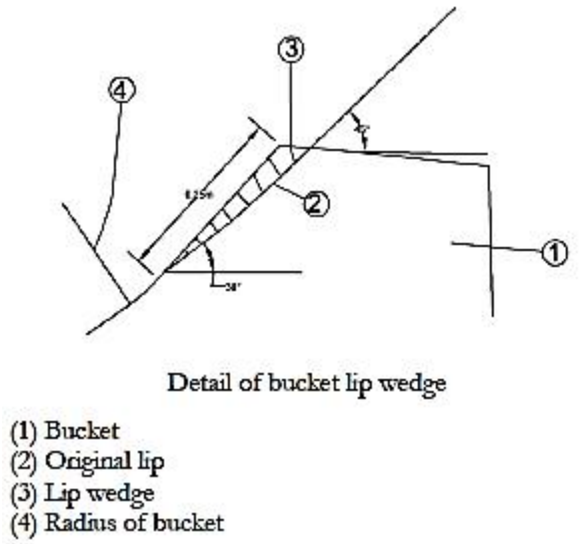


Figure B.1- 17 Guri Spillway Model showing deep scour in the right side of the Energy Dissipater, loss of the right corner of the Chute 2 of the spillway, loss of walls 2 and 3 in the bucket area. (EDELCA)

The Expansion - Guri Final Stage Project

By the time of finishing Guri First Stage Hydroelectric Project, there started the expansion of the Hydropower Station by building a second powerhouse with 8000 MW, with its tailrace channel a gravity dam and raising the existing spillway and the existing gravity dam by 55m. Moreover, two 140 m high embankment dams were built to close the dam site at the left and right abutments. A total of 7,100,000 m^3 of concrete were needed to expand the project and 90,000,000 m^3 of fill materials to complete the earth-fill and rock-fill dams (Figure B.1-18 and B.1-19). Guri Final Stage Project is probably the largest dam raising project ever where most challenging engineering aspects were developed and reported.

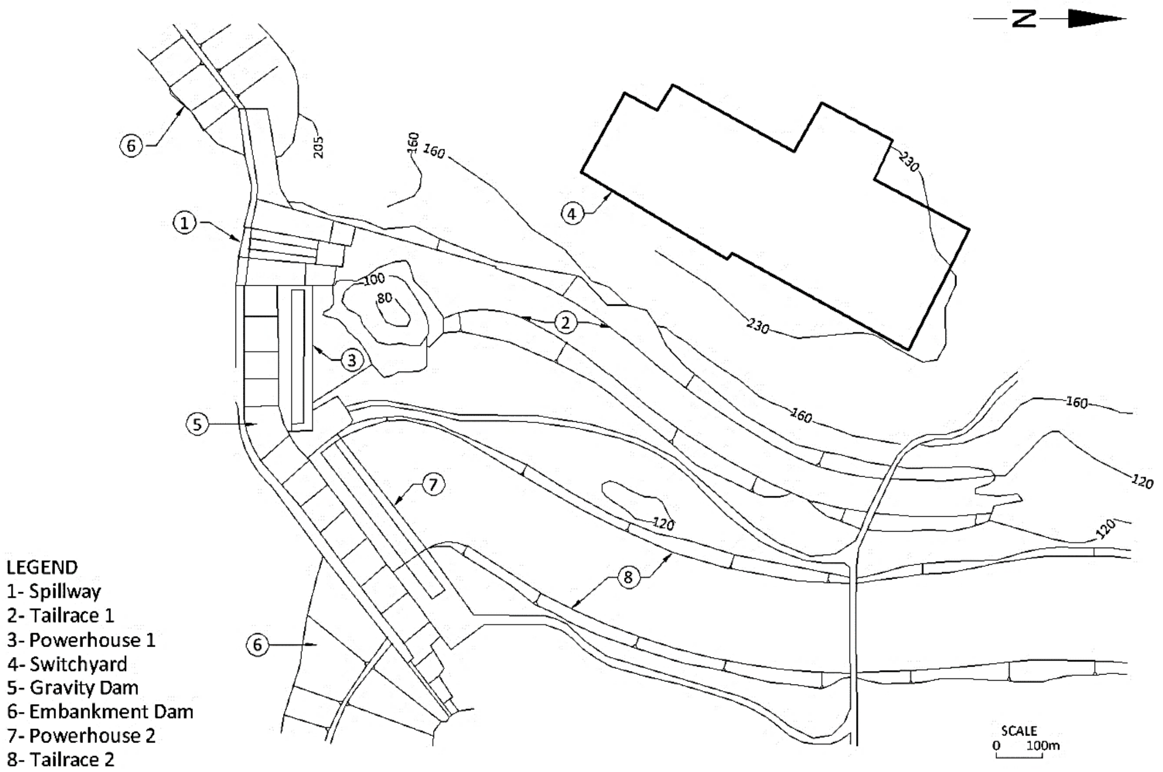
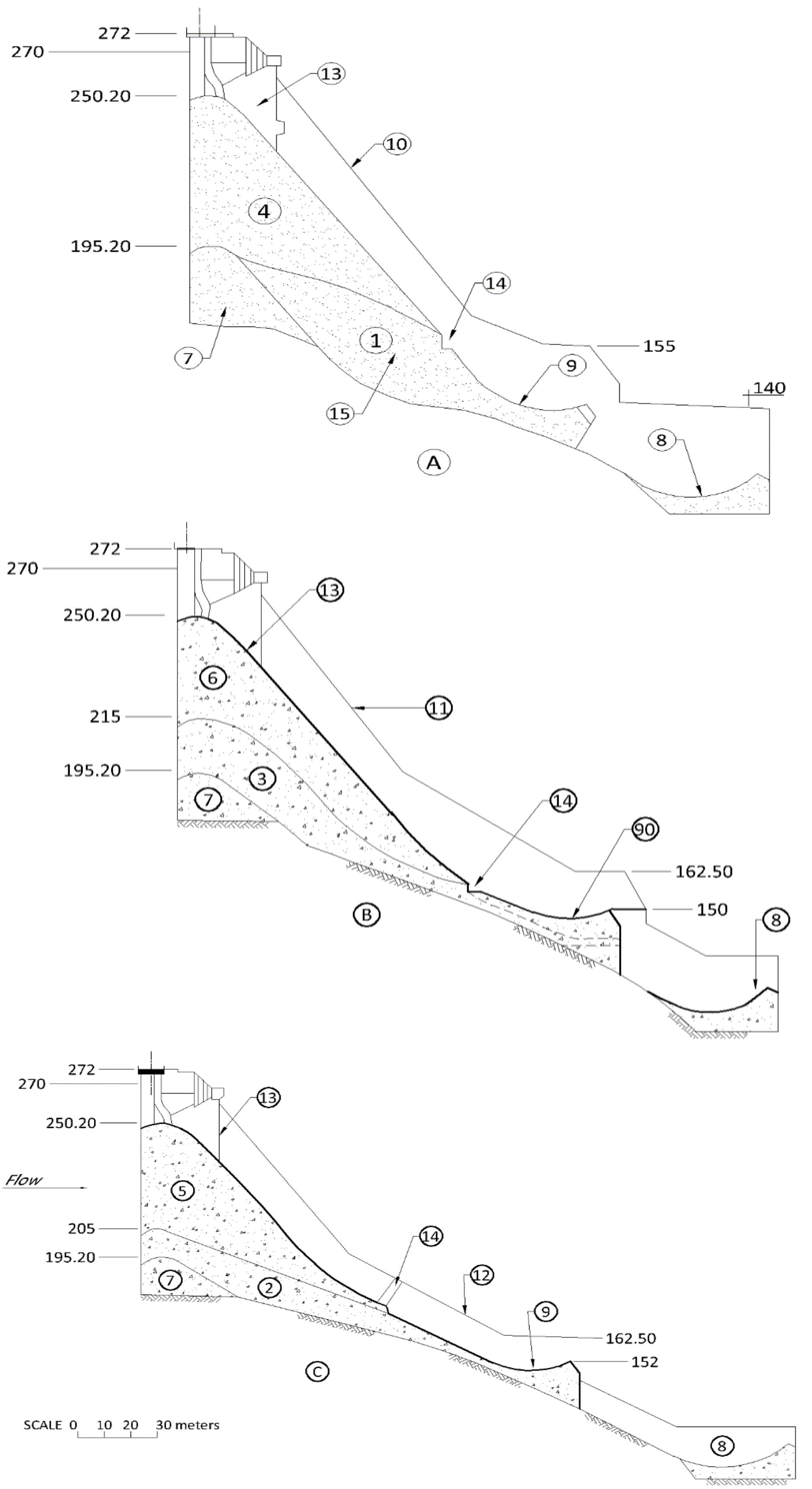


Figure B.1-18 Guri Final Stage project main works. (Marcano, et. al., 1988).

Raising of the Spillway of Guri Dam

Guri Spillway was raised in 6 stages for a total of 55m (Figure B.1-20). Reservoir area and volume increased to 3,850 km^2 and 140,000 x 10^6 m^3, respectively. During each construction stage, concrete was poured in only one chute while the two others had to be ready to pass the excess water. It was also decided to raise the flip buckets that were a major change in the spillway and in the project. Incorporating of aeration devices was also included in the Final Stage of construction to prevent the cavitation damage and improve the performance of the spillway.



Legend

A – Chute 1

1- Stage I
4- Stage IV
7- Original
8- Original Flip buckets
9- Raised Flip buckets
10-Training Wall 2
13-Gate Pier
14-Lower aerator
15-Concrete Stage I

Legend

B – Chute 2

3- Stage III
6- Final Stage
7- Original
8- Original Flip Buckets
9- Raised Flip Buckets
11- Training Wall 3
13- Gate Pier
14- Lower aerator

Legend

C – Chute 3

2- Stage II
5- Final Stage
7- Original Chute 3
8- Original Flip buckets
9- Raised Flip buckets
12-Training Wall 4
13-Gate Pier
14-Lower aerator

Figure B.1-19 Guri Final Stage Spillway. Process of raising the Spillway and flip buckets.(EDELCA)

Figure B.1- 20 Process of raising the spillway by stages. (EDELCA)

Design criteria for raised flip buckets

Geometry and performance of the flip buckets of the 3 chutes was extensively physically model tested in the 3d Comprehensive Physical Model built in EDELCA Hydraulic Laboratory. The following criteria were considered for the design of the energy dissipaters:

1. Distance of thrown of the jet was limited by the existing plunge pool, that is no need for produce additional erosion downstream.
2. Geometry to produce a jet to convey the jet to the existing plunge pool
3. Lip angle to reach the existing pool and, to provoke erosion in the downstream direction and sedimentation upstream to fill the caverns left around the original flip buckets
4. Piezometric pressures be maintained positives, in the worst case, tolerate negative pressures but around atmospheric pressure at the bucket lip
5. Use lateral ramps, to induce jets to fall in the existing pool area
6. Guarantee free jets performance even for the PMF condition
7. Radius of the bucket should be between 3-5 the flow depth

8. Flip buckets and aerators located upstream, should be compatible and avoid flow modification entering the flip buckets due to aerator performance

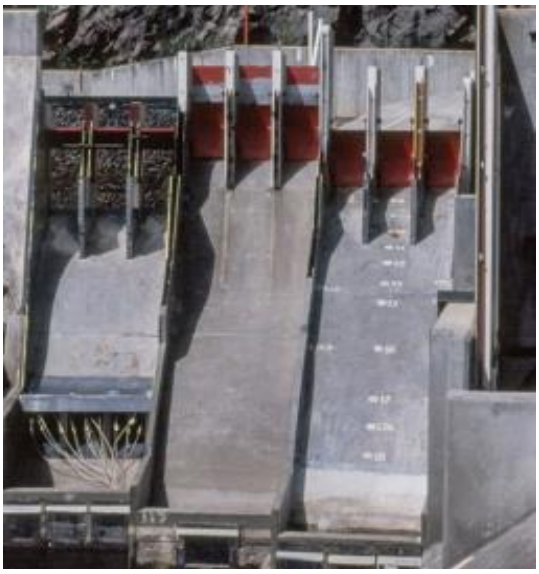


Figure B.1- 21 Model Scale 1:50 during the tests for the flip buckets of Chute 1 (Stage1). Note that Chute 2 and 3 shows original flip buckets and crests of the 3 chutes are at different elevations. (EDELCA)

Figure B.1- 22 Physical Model scale 1:50 showing the performance of the 3 raised flip buckets'. Note that aerating devices are also incorporated to the spillway model. (EDELCA)

Selected geometry for the flip buckets

A great number of geometries for the energy dissipaters of the 3 chutes were tested in the Physical Model Scale 1:50, both during the construction stages and the final stages. Flip buckets were built in Stage 1, 2 and 3 for Chutes 1, 3, and 2, respectively (Figure B.1-23). In the same figure, resulting geometries for the final stage are shown together with original dissipaters. On other hand, Figure B.1-24 shows performance of the 3 raised flip buckets in a Physical Model scale 1:50.

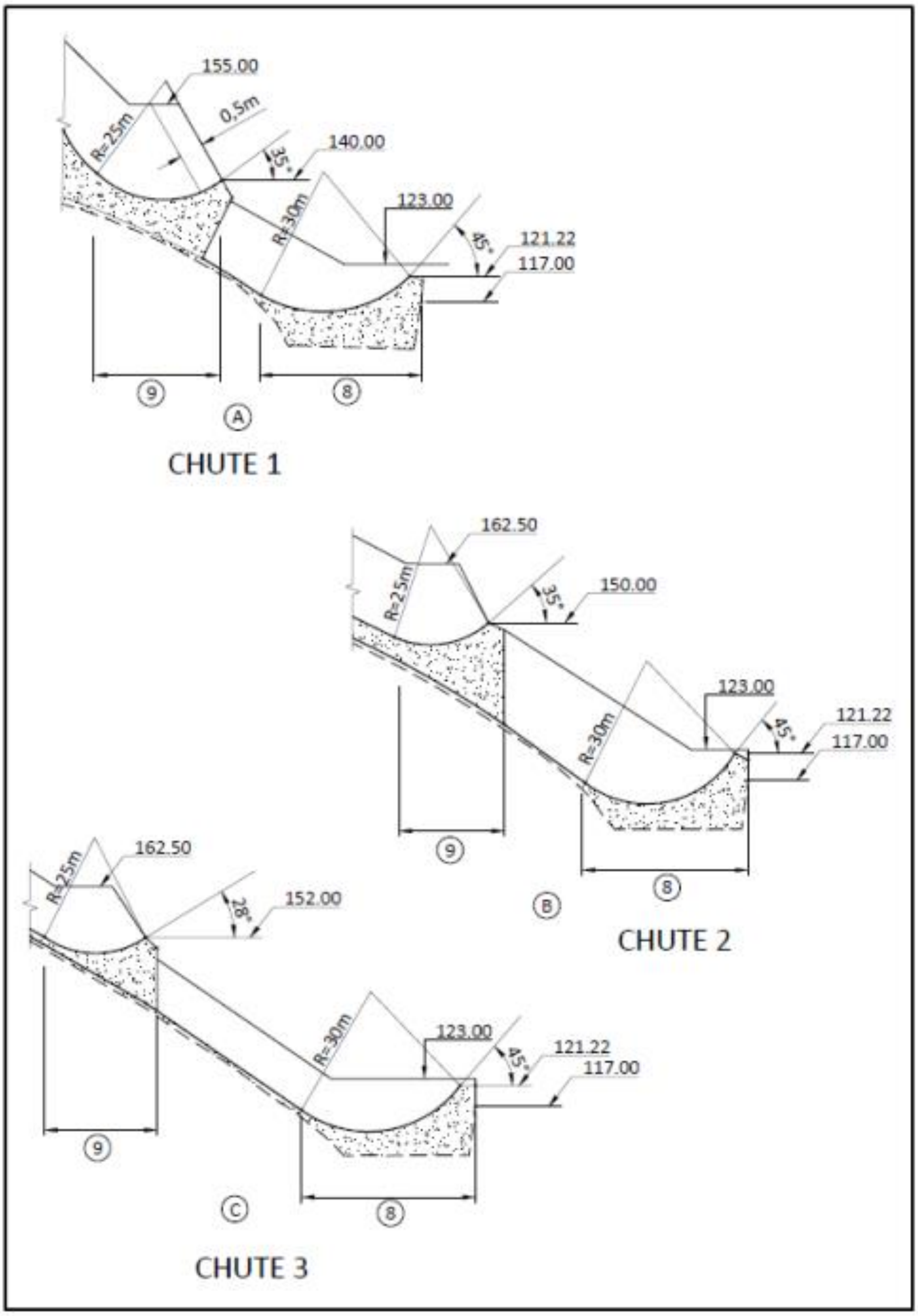


Figure B.1- 23 Selected geometries for Chutes 1, 2 and 3 of Guri Spillway. (Marcano, 1999)

Performance of the raised flip buckets

Chute operation was very prolonged and diverse, following the schedule of construction where headwater was changed from El 215 to El 270 m. Chute 2 maximum spillway discharge during the construction stage was 14,250 m^3/s. The flip buckets of Chute 1 were raised in 1979 and were operating for more than 100 days with flows higher than 10,000 m^3/s. The flip buckets of Chute 3 and 2 were raised in 1981 and 1982, respectively (Figure B.1-25 shows final stage buckets). Since then, all three flip buckets have operated alternatively for discharge of 9,000 m^3/s per chute. Scouring took place moderately, and the plunge pool developed with scoured volume of nearly 400,000 m^3 in the first 10 years of operation (Figure B.1-26). Development of the plunge pool resulted as expected, with scouring and filling downstream and upstream, respectively (Figure B.1-24). Resulting envelopes of the boundaries of the plunge pool can be seen in Figure B.1-25.

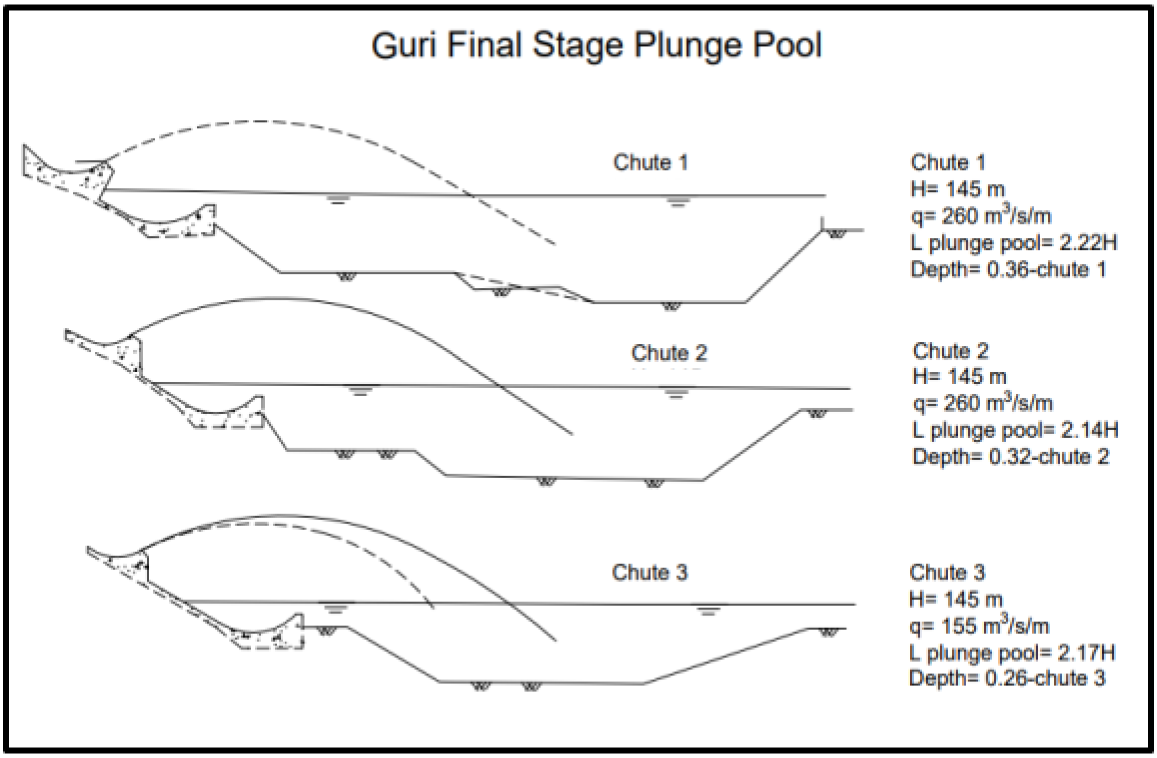


Figure B.1- 24 Envelope of Idealized Guri Spillway Plunge Pool (Marcano, 1999).

Lessons learned

- Operating conditions of large structures are of some uncertainties, and for such high energies, design should be conservative.
- Regular visual inspection of these structures is fundamental after each flood operation. That was not possible at Guri original flip buckets.
- Repairing works relying on the spillway walls and bulkhead were so difficult with complicated logistics.
- Availability of the physical model was a fundamental advantage to solve the difficult change in the design and during the construction of the works.
- Use of training walls helped to operate the spillway during the complex process of raising the spillway and flip buckets.
- Submerged large energy dissipaters of Guri First Stage operated randomly.

Aeration at Guri Final Stage Spillway

Figure B.1- 25 Chutes 1 and 3 of Guri Final Stage Spillway operating (EDELCA)

Guri Final Stage (Figure B.1-25) started construction in 1978 to raising the spillways by 55 m in 6 stages. Reservoir area increased from 850 Km2 to 3,850 Km^2 with a new total storage volume of 140,000 x 10^6 m^3, respectively. During each construction stage, concrete was poured in one chute while the two others had to be ready to pass the excess water. It was decided to raise the flip buckets that were a major change in the spillway and in the project. Incorporating the aeration devices to prevent the cavitation damage was also done in parallel at different stages and completed in the Final Stage of construction. The provided aeration improved the performance of the spillway.

Original experiences of Aeration at Guri Spillway

First experiences of artificial aeration at Guri Spillway started in 1980 in the Guri Comprehensive 3D Physical Model (Figure B.1-26), where a set of ramps of different height and exit angle were tested for the original geometry of Chute 3. These ramps were located at the end of the spillway trestle piers. These ramps were accompanied by lateral wedges built at each side of the trestle pier in the physical .These wedges reduce the impact of “rooster tails “which tend to perturb the air inflow into the air cavity.

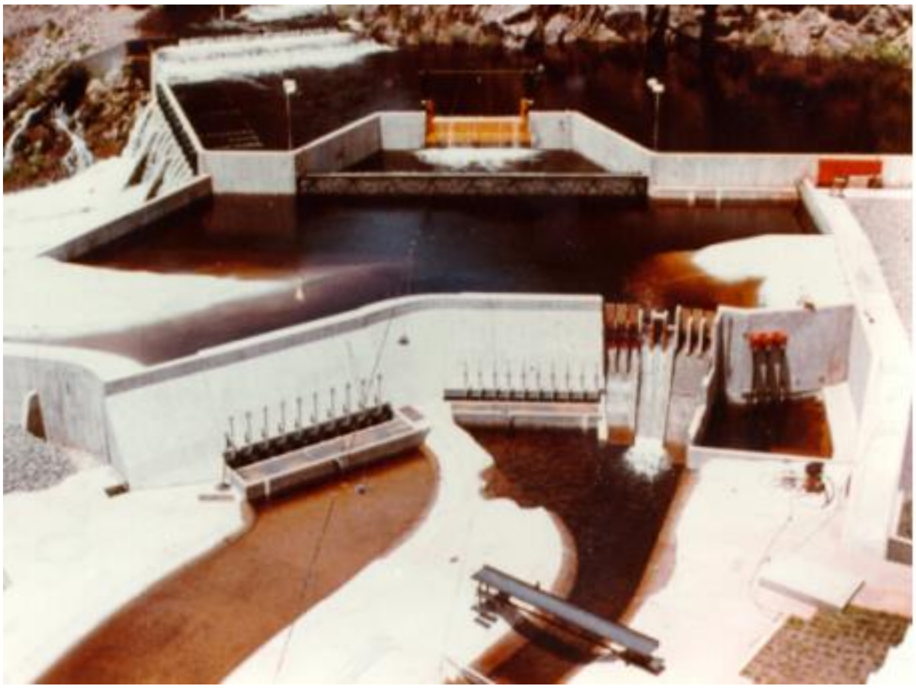


Figure B.1- 26 Comprehensive 3D Physical model of Guri Final Stage Project (Courtesy of EDELCA)

Model tests allowed selecting two ramps to be tested in the prototype between 0.40 m and 0.60 m height. A scale was painted on the walls and on the spillway floor to measure the air cavity length. Each ramp was instrumented with piezometers with the purpose of measuring the air pressure along the ramp and evaluates from these measurements the capacity of aerator to introduce air behavior inside the water nappe. The lateral wedges were included in the prototype (Figure B.1-27). A first ramp 0.40 m height was built and tested in the bay 8 of Chute 3. These tests did not give good results. A ramp 0.75 m high and 17 ° of exit angle was selected and built in the 3 bays of Chute 3.

Figure B.1- 27 Prototype ramp 0.75 m, 17° exit angle built in the 3 bays of original Chute system of measurement of the Cavity length and piezometric taps. (EDELCA)

The 0.75 m ramp gave promising results. Gate was opened from 1 to 10 m, cavity lengths and piezometric pressure measurements were taken (Figure B.1-28, B.1-29 and B.1-30). Negative pressure measurements were as low as –0.25 m of water column and cavity length was about 35 m, which is considered sufficient for representing a good air flow inside the cavity. The cavitation damage that was present in the lower reaches of chute 3 was arrested and Chute 3 continued operating satisfactorily with the upper aerators after the rainy season. The artificial aeration method was approved to be adopted for all the intermediate stages of raising the Guri Spillway.

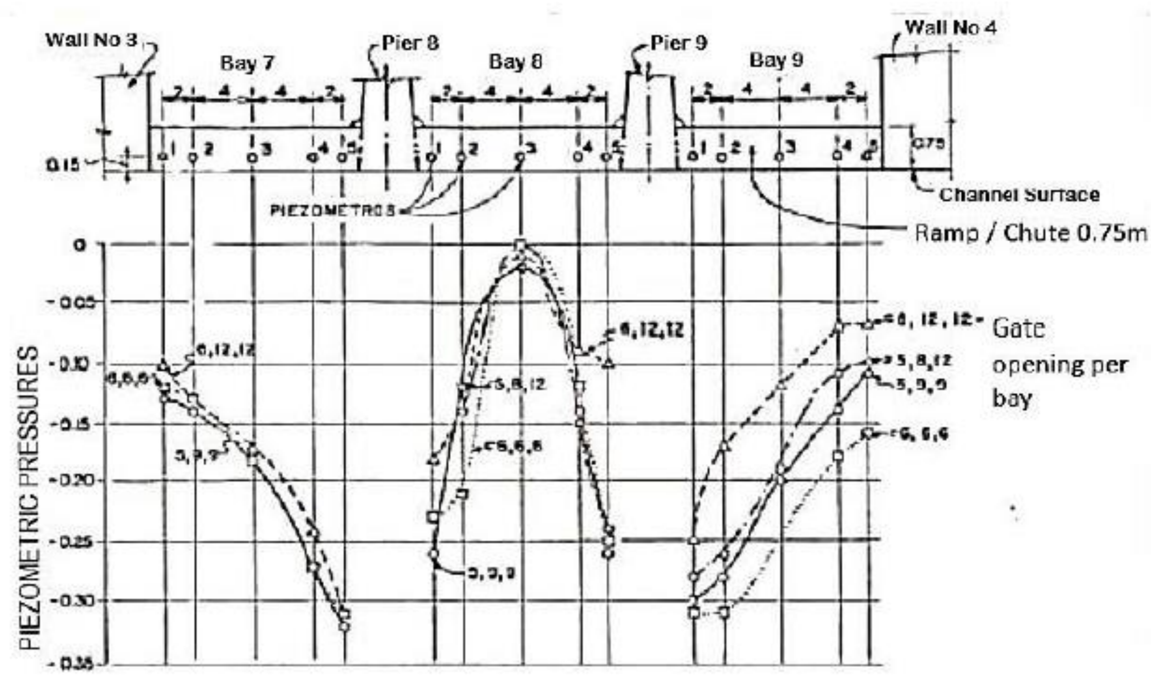


Figure B.1- 28 Piezometric pressures at the face of the ramp (prototype tests) inside the air cavity (EDELCA, 1987)

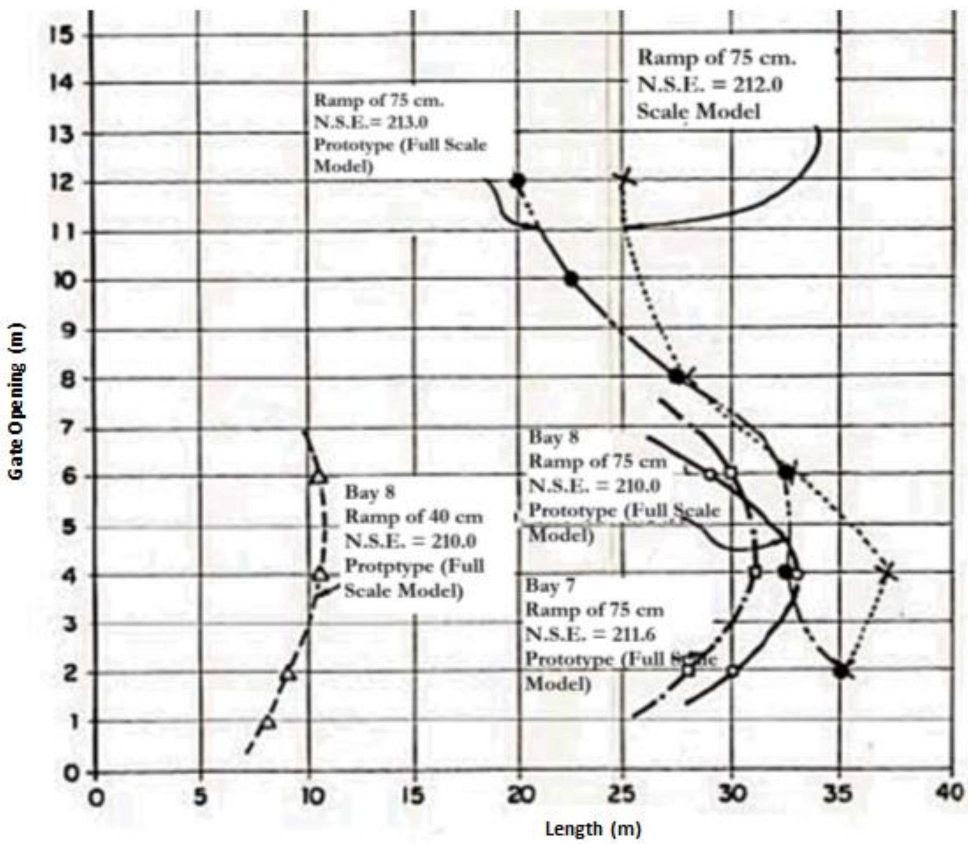


Figure B.1- 29 Prototype measures of the Cavity Length for different ramps tested in the proto-type and in the model (EDELCA, 1987)

Figure B.1- 30 Prototype tests on Chute 3 Original for measurements of cavity Length

Design Criteria adopted for the selecting the geometry of the aeration devices for Guri Final Stage Spillway: The following criteria, compiled from different sources of limited number of aerators in operation at that time included (Prusza et alia, 1982).

Guidelines – On the aerator

1. The most used types of aerators were ramps and steps or combination of both. The mechanism of air inflow is improved when a small ramp is placed on top of the aerator (Zagustin et alia, 1983)

2. Selection of the aerator is determined by conditions and design characteristics of the spillway, the spillway geometry, and guarantee of a good performance of the source of supply of the air into the cavity

3. Dimensions of the aerator should be obtained from physical Model, of adequate scale, following the relationship between the air cavity length and the flow depth at the ramp lip.

4. During the physical model tests, several problems should be studied: blockage of the air cavity by water, obstruction of the cavity by rooster tails, waves, or any other flow effect, interaction of flow from the cavity with the geometry of the spillway, and other aspects that are very difficult to predict by simple computations

5. When there is a curve surface in elevation, the aerator should be placed in the preceding straight reach. For a same step, a larger angle ramp will produce longer air cavities. Also, for the small gate openings there will be very long air cavities. The cavity length in general should not be 20-25% longer than the cavity length for the maximum water depth.

6. The type of aerator to select will depend on the amount of air needed to protect the concrete surface at that station in addition to the step and the ramp placed on top of the step. It there is a need to increase the volume of air into the flow, the use of lateral ramps at each wall side may increase the air volume entrained. The placing of lateral wedges will also serve as to improve the amount of air to be entrained in the cavity. The dimensions of the wedges are the same as the dimension of the ramp forming a 45° at the end of the aerator. These wedges improve the air inflow process eliminating or reducing rooster tails or any other flow perturbation that may make difficult air entrainment in the cavity.

Guidelines-On the air concentration

1. The minimum concentration of air in the mixture air/water should be 7% - 8% calculated by the expression C=Qair/(Qair+Qwater), where C is the concentration of air (%) Qair is the air volume in m^3/s, Qwater is the water flow in m^3/s, corresponding to the boundary layer at the aerator lip.

2. The air concentration produced at the system should not be less than:

   C=8+L x c, where "C" is the air concentration (%) "L" is the spillway length (m) and "c" is the air concentration loss per liner meter, (%) that varies according to the following:

   0.15-0.20%-for straight reaches
   0.50-0.60% for concave geometries
   0.15-0.20% for convex geometries

3. The maximum air concentration to introduce in the flow is 45%. For higher concentrations the flow pulverization occurs.

4. To obtain an air concentration of the order of 40-45%, the cavity length produced by the flow at the aerator should be approximately between 4-5 times the flow depth at the aerator lip

5. Under pressure in the air cavity should not be less than -1m of water column (1m of negative pressure)

6. When two aerators are needed, it should be introduced the larger amount of air in the upper aerator and the second one acts as a supplement.

7. When there are more than one aerator, the distance of the new one varies in the range 60-80m and it is adapted to the spillway layout

Guidelines-On the air supply source

1. When the air supply system is by using galleries and air ports, the total area of the air ports should be similar or higher than the main gallery area. Air ports should be placed equidistant from the intake galleries in such a way that the air enters in the most uniform distribution along the water flow and be distributed to the total width of the spillway.

2. When the air is supplied through galleries, air velocities inside the galleries should not exceed 100-120 m/s.

Process of design of the aerators for Guri Final Stage Spillway

To define the aerators needed for the final stage spillway profile, a number of investigations were made in the model and prototype. Tests of aerator geometries were selected to provide enough air to the water flow, in conjunction with the construction program me and the time of operation of a given stage of construction. Through this process, it was possible to design he different aerators that became into operation as the raising of the spillway was progressing. From many alternatives of aerators, 9 aerators were designed and tested in the prototype.

Instrumentation for model Investigations

In addition to the traditional laboratory instrumentation, the under pressure in the cavity (Figure B.1-31) was artificially simulated, where it was possible to simulate pressures as lows -4 m of water column to evaluate the collapse of the air cavity.

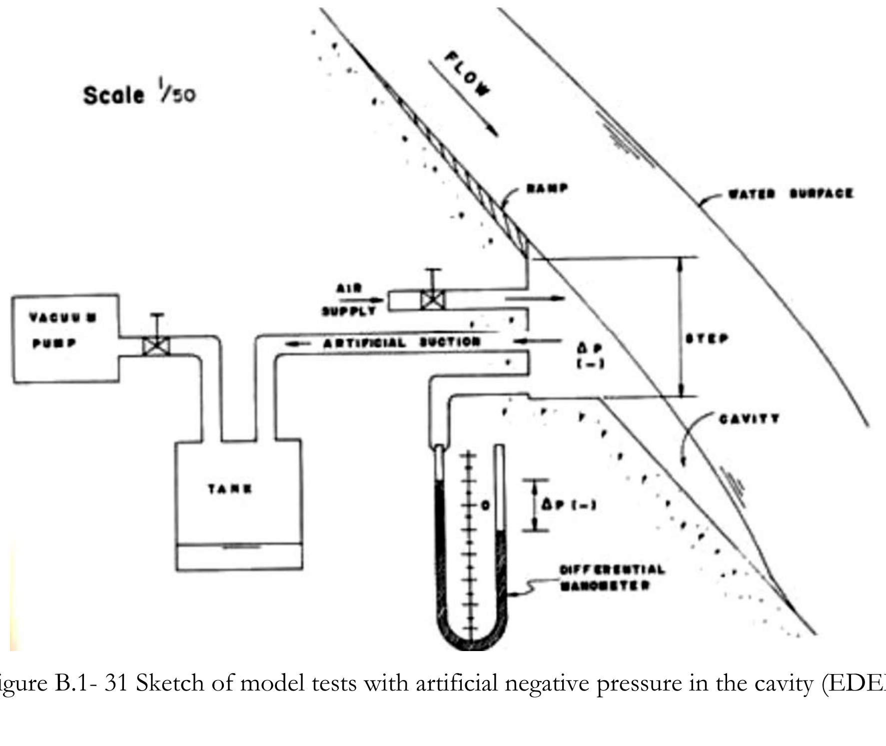


Figure B.1- 31 Sketch of model tests with artificial negative pressure in the cavity (EDELCA,1987)

Prototype Instrumentation

To understand the behavior of the air supply system in the prototype, the different aerators and system of air supply were instrumented. In both the aeration cavity and in the galleries/ air passages, piezometers and air anemometers were placed at selected locations. See Figure B.1-32.

Piezometers

A number of piezometric orifices were located to measure the air pressure in the different air passages, including the air pressure in the face of each aerator, in the air ports, in the galleries for each water condition as the aerators were coming into operation. Piezometers consisted in cupper orifices of ¼ inch located strategically in different points of the system aerator-air supply system.

Anemometers

A set of anemometers were located the different air passages in order to measure the air velocity and then the air demand in the different locations. The anemometers permitted to obtain the air flows, at known water flows, and then the air concentration in the boundary layer could be obtained. The anemometers used were of German manufacturer, "Honzsch" for air velocities up to 120 m/s.

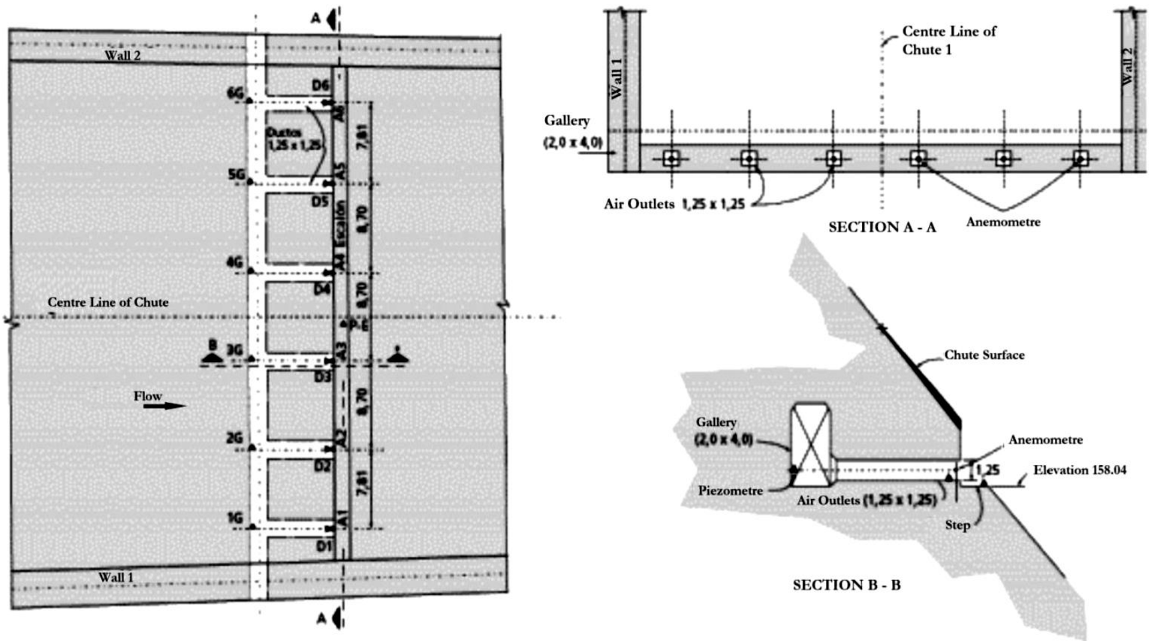


Figure B.1- 32 Prototype Instrumentation for Chute 1 (EDELCA, 1987)

Chute 1 Stage 1 aerator

The first aerator to operate on a new concrete chute surface which was built for the project expansion was located in Chute 1 (Figure B.3-8). It consisted in a 2m step height with 1,434m of base, with a 0.10m ramp above the step with a horizontal base of 2,49m. The air supply system for Chute 1 consists of 6 air ports 1,25x1,25m connected to a gallery 4x2m with air intake at Wall 1 entering from the spillway wall 1. Measurements taken for different gate openings from 1m to 18m revealed that under pressure in the air cavity was as low as -0,8m of water column, well within the -1m of water column selected as design criteria. With base in the air discharges calculated, an air concentration between 30 and 45% was estimated within the boundary layer, which corresponded well with the expected results. Chute 1 stage 1, operated for 2 rainy seasons with large flows including 10,600 m^3/s for 60 days (Marcano, 1999). Damage in the spillway surface was not observed which proved the efficiency of the aeration system.

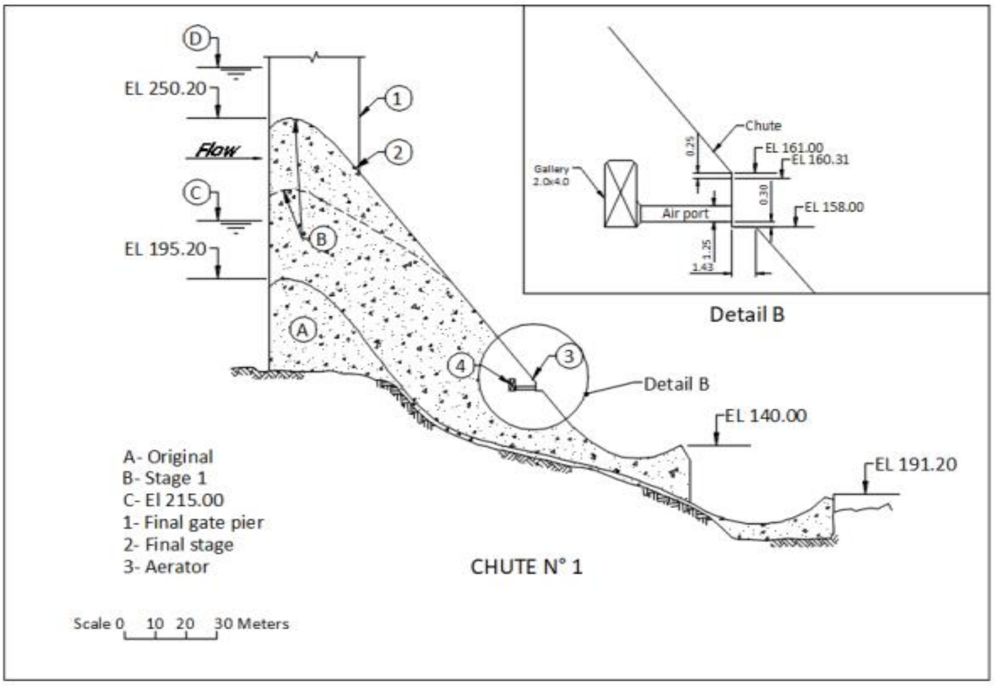


Figure B.1- 33 Guri Final Stage Spillway. Aerator for Chute 1 Stage1- (EDELCA, 1987)

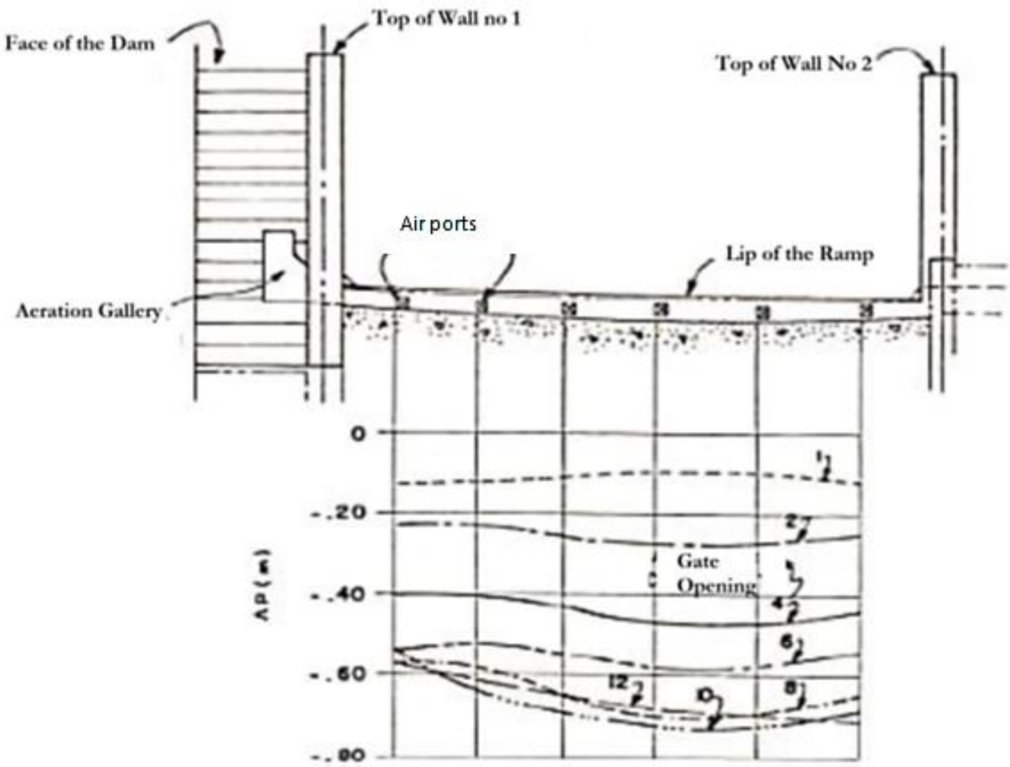


Figure B.1- 34 Under pressures distribution inside the air ports (EDELCA, 1987)

Figure B.1-34 shows air distribution in the air ports and Figure B.1-36 shows air concentrations during water flow (Zagustin et al, 1985) Chute 2 Stage III aerator

Chute 2 Stage III aerator

In Chute 2, the aerator consists in a step of 2.0m step and a ramp 0,25m heigh x6.4° of exit angle. The air supply system consists of two main galleries that take the air from lower part of the flip buckets vertical wall. These two galleries join a main 3.1x2 m gallery where the air is distributed to 6 air ports 1.25x1.25m. (Figure B.1-35).

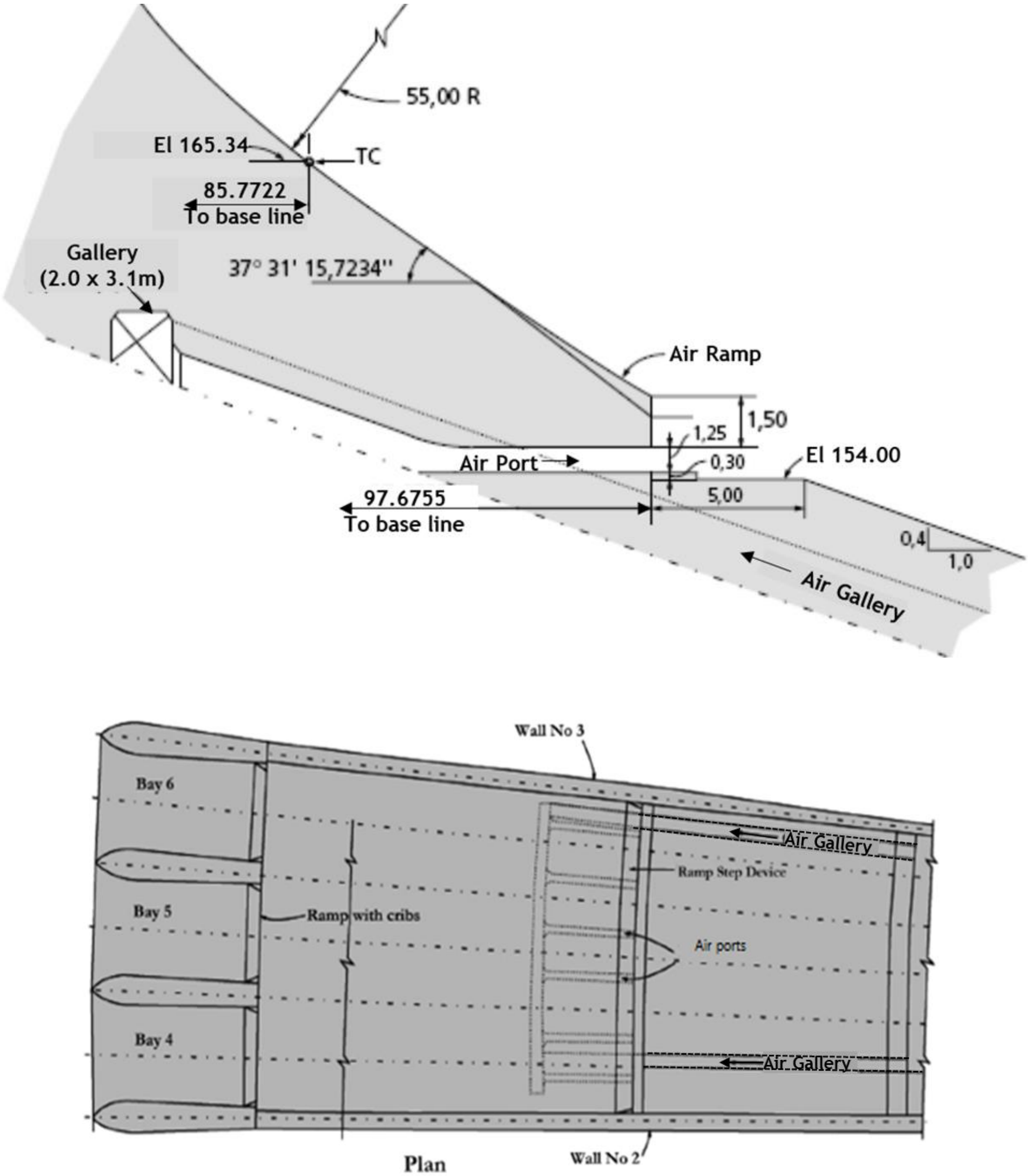


Figure B.1- 35 Lower aerator for Chute 2 Stage III (EDELCA, 1987)

Chute 3 Stage 2- prototype experience

Model tests for Chute 3 Stage 2 indicated the need to build an aerator consisting of a ramp 0.15m with 2.5° with lateral wedges and lateral ramps of 0.30m x 5°. As air supply source for Chute 3 Stage 2, a slot 2mx 0.75m depth, Chute 3 started operations in the rainy season of 1982 and with only 190 hours of operation at discharge of 2,000 m^3/s, it suffered severe cavitation damage immediately downstream of the aerator and some damage in the central part of the flip bucket. The damage was concentrated at the axis of the Chute, covering some 2.5m long section in the flow direction and a maximum scour hole of about 0.30m (Figure B.1-36 and B.1-37).

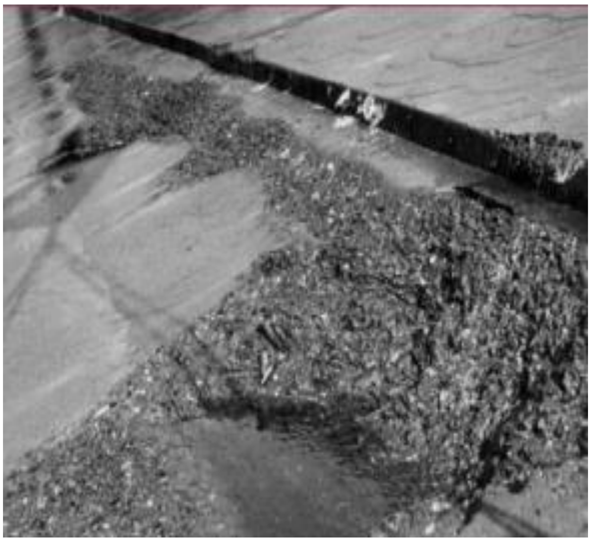


Figure B.1- 36 and Figure B.1- 37 Chute 3 Stage 2-Cavitation damage after 190 hours of operation with 2,000 m^3/s (EDELCA)

The chute was repaired with shotcrete and the face of the aerator was instrumented with piezometers. A set of prototype tests were done, and the results showed very low under pressures, as low as -7m of water column (Figure B.1-38). These very low values represented cavitation conditions to occur because of the difficulty of the air flow to travel from the walls to the center of the chute that was some 50m wide at the aerator station. Actually the same damage appeared once again after some hour of operation.

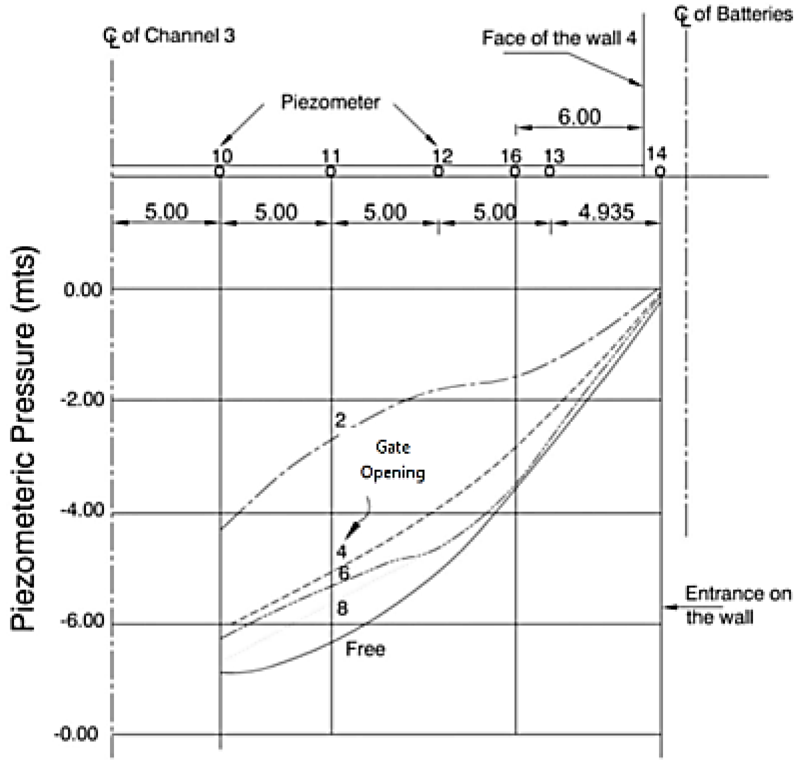


Figure B.1- 38 Piezometric pressures measured in Chute 3 Stage 2 (Patiño et al, 1987)

A new ramp 0.40m height x 7° was selected in the Physical Model Esc 1:50. The ramp was built and tested. Piezometric pressures registered in the prototype tests were as low as- 1,8m of water column, which was considered reasonable for a temporal condition. Results of the prototype tests are shown in Figure B.1-39.

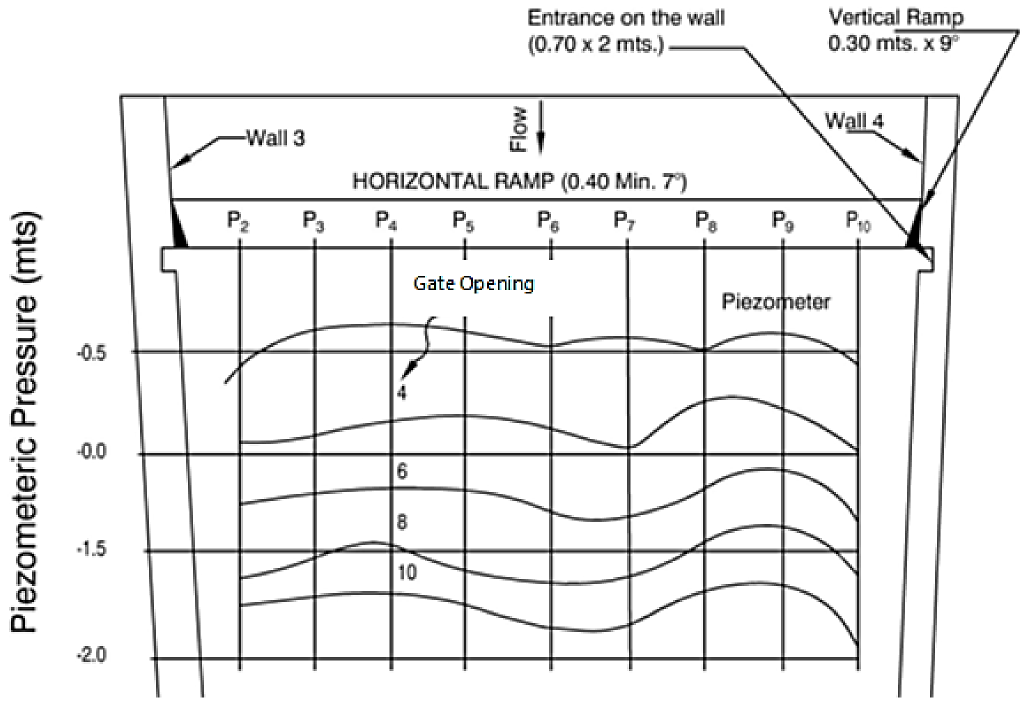


Figure B.1- 39 prototype piezometric pressures taken at the 0.4 m x 7° ramp built in Chute 3 Stage 2. (EDELCA)

Aerators for the final stages

Aeration ramps were selected from Physical Model tests in the Guri Comprehensive Model Esc 1.50. Model aeration devices were tested with under pressure beneath the air cavity. From the prototype experience it was also observed the capacity of the air systems at Chute 1 and Chute 2 of providing 45% or air concentration, required to obtain the adequate protection against cavitation damage. A final selection of the aeration system for Guri Final Stage Spillway resulted in the following devices.

Upper aerators

For the 3 chutes, a similar aerator was designed and placed at the rear of the gate piers. The aerator is 1m height x 7° angle with lateral wedges of 1x1m, the air supply system is obtained at the rear of the piers (Figure B.1- 40). Limited prototype measurements done after filling of the reservoir to final stage level indicated under pressures of -0.25m of water column and air velocities entering the air cavity of 22.5 m/s for a gate opening of 4m. Figures B.1- 41 and B.1-42 also shows the way air enters the water flow at the rear of the piers.

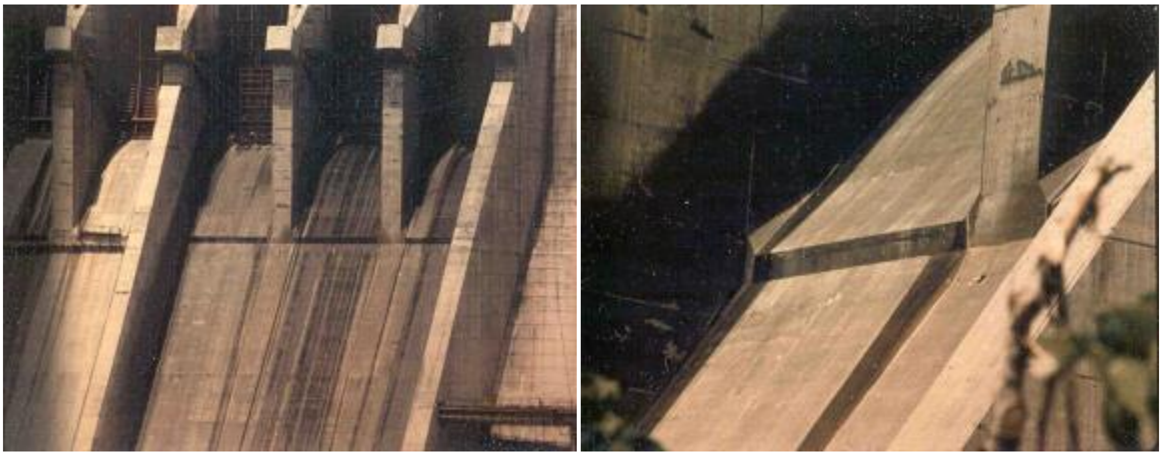


Figure B.1- 40 Guri Final Stage Spillway. Upper aerators for Chutes 1,2 and 3 (EDELCA, 1988)

Figure B.1- 41 Guri Final Stage spillway. Upper aerator. Air intake at the rear of piers (EDELCA, 1987)

Figure B.1- 42 Flow from the upper aerator, See the air emerging from the air cavity once air flow in entrained at the rear of the gate piers (EDELCA, 1987)

Chute 1 Final Stage Lower Aerator

For Chute 1 Stage 1, the lower aerator resulted in a step 2.84m with a ramp of 0.25mx 5° exit angle. Air is supplied through the main gallery 2x4m connected to 6 airports 1.25x0.25 m. Prototype tests executed at final reservoir level permitted to obtain the data of under pressures in the cavity, air flows and air concentration and were compared with the previous results obtained from the prototype measurements for Chute 1 Stage 1 (Figure B.1-43). Tables B-1.1 to 1.3 also presented the most relevant parameters of prototype performance for the two stages of Guri Spillway.

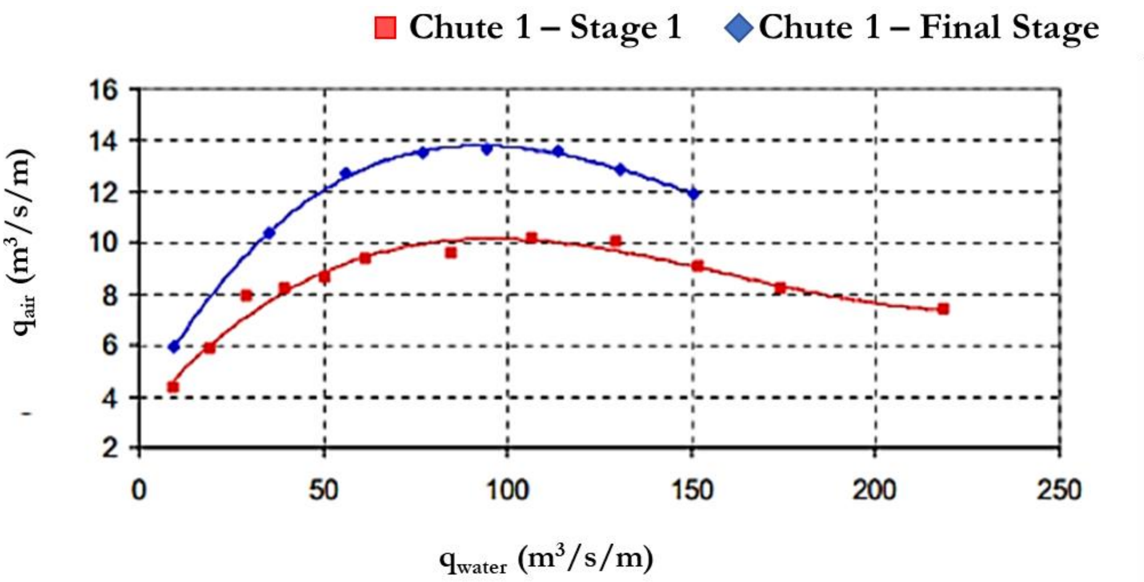


Figure B.1- 43 Chute 1 stage 1 & Guri Final Stage Project. Air flow vs. water flow (EDELCA,1987)

q_W (m^3/s/m)	9.47	35,39	6,17	6,95	4,65	13,58	30,66	150,62
C (%)	51,03	59,39	62,86	61,74	61,07	59,92	57,95	53,71

Table B.1 1 Air concentration vs. water for Chute 1 Final Stage (EDELCA, 1987)

qw (m^3/s/m)	9,26	9,34	9,42	9,92	0,62	61,73	84,77	107,00	129,63	152,26	174,49	218,93

C (%)	31,78	34,47	40,77	41,20	41,79	49,34	44,03	45,41	44,73	42,10	39,37	36.69

Table B.1 2 Air concentration vs. water for Chute 1 Stage 1 (EDELCA, 1987)

Stage	Reservoir El (masl)	Average Under pressure (m water column)	Cmax (%)	V air (m/s)	q max air ($m^3/s/m$)
Chute 1 Stage 1	270	-0,70	49,34	80	10
Chute 1 Final Stage	215	-1,40	62,86	96	14

Table B.1 3 Comparison of prototype values for Chute 1 Stage 1 and chute 1 Final Stage (EDELCA, 1987)

Figure B.1- 44 Chute 1 Final Stage. Lower and upper aerators. The airports, the step and the small ramp on the step in the lower aerator can be seen. The upper aerator is also visible. (EDELCA)

Chute 2 Final Stage Lower Aerator

For Chute 2 Final Stage1, the lower aerator resulted in a step 2.2m with a ramp of 2.75m height x 25° exit angle (Figure B.1-44). Air supply is through two main galleries with air intake at the downstream face of the chute 2 flip buckets. These two galleries are connected to a distribution gallery 3,1x2m that distributes the air to 6 airports of each 1.25x1.25m. Prototype tests executed at final reservoir level permitted to obtain the data of under pressures in the cavity, air flows and air concentration and were compared with the previous results obtained from the prototype measurements for Chute 2 Stage 3 (Figure B.1-45); Table B.1-4 also presented the most relevant parameters of prototype performance for these two stages of Guri Spillway. The fact that under pressures are lower than the under pressures measured for Chute 1, may be due to a more restricted air flow passages in Chute 2.

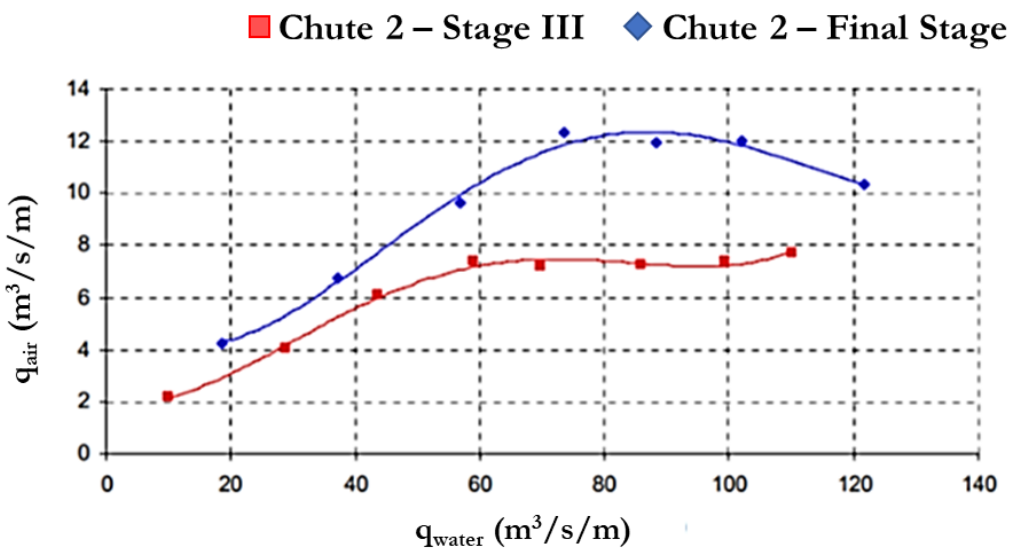


Figure B.1- 45 Chute 2 stage 3-Chute 2 Final Stage. Guri Final Stage Project. Air flow vs. water flow (EDELCA, 1987)

Stage	Reservoir El (masl)	Average Under pressure (m water column)	C (%)	Air(m/s)	Q max air (m^3/s)
Chute 1 Stage 1	270	-1,20	35	6	378
Chute 1 Final Stage	215	-2,50	58,33 (8m)	3	612

Table B.1 4 Comparison of prototype values for Chute 1 Stage 1 and chute 1 Final Stage (EDELCA)

Chute 3 Final Stage Lower Aerator

For Chute 3 Final Stage, the lower aerator resulted in a 0.80mx12.08° exit angle provided with a lateral ramp of 0.3mx5° and a wall slot 2 m x 0.75 m in depth in walls 3 and 4 (Figures B.1-46 to B.1-48). These slots have performed well during the releases of Chute 3. No evidence of cavitation dam-age has been observed in Chute 3.

Figure B.1- 46 Chute 3 Final Stage. Lower aerator. Note the ramp, the step, the lateral ramp and the wedge. (EDELCA, 1987)

Figure B.1- 47 Chute 3 Final Stage of Guri Spillway. Lower aerator. (EDELCA, 1987)

Figure B.1- 48 Chute 3 Final Stage of Guri Spillway. Lower aerator in operation. (EDELCA)

Lessons learned

1. At Guri Final Stage project, it was possible to improve the original project. Flow aeration was incorporated with great success even for such a large structure with an intensive operation.
2. Combined prototype and physical model investigations were key in obtaining the best results for the Final Stage project (see Figures B.1-49 to B.1-51).
3. At a 1:50 Scale physical model, it was impossible to reproduce the air volume in the water flow, but the geometric conditions of the aerating devices were correctly simulated. It was also possible to measures the cavity and the artificial under pressure was very useful in obtaining the limits of the operation of each aerator.
4. Prototype measurements were of invaluable benefit. It permitted to test the intermediate stage aerators and improve the design for the final stage devices.
5. The use of the physical model proved fundamental during the process of defining the final geometry of the aerators.
6. The system of supplying air through air galleries selected on the basis of the proper architecture of the existing structure showed the best results for the prototype structure.
7. The system of aerating large spillways by using galleries was found necessary due mainly to the large specific flow and depths of discharge over Guri Spillway (i.e. unit flow of 250 $m^3/s/m$ and depth 7m to 8m)
8. Providing artificial under pressure in the aerator air cavity is of great help to assess the capacity of the cavity that collapses with under pressures up to -4m of water column.
9. Monitoring of concrete surface of the spillway should continue as in spite of good aeration other effects may take place in the old structure such as chemical reactions.

Figure B.1- 49 Guri Final Stage Spilling with a very high content of air (EDELCA)

Figure B.1- 50 Guri Spillway. Chute 1 Final Stage in Operation. Note prominent bulking of flow. (EDELCA)

Figure B.1- 51 Guri Spillway. Chute 1 Final Stage in Operation. Note prominent bulking of flow. (EDELCA)

Macagua Hydroelectric Project

Macagua Hydroelectric Project is a part of the Hydroelectric Development of the Lower Caroni, located at the South-east Venezuela and has been in operation for 22 years. It consists of a CFRD dam of maximum height of 35 m, a spillway with capacity of 30,000 m^3/s, equipped with 12 gates 22m width x16m height each and 3 Powerhouses of 380, 2400 and 100 MW each, for total of 2,980 MW . The spillway is located upstream of La Llovizna falls, a local tourist attraction; it dissipates its energy directly over the river bed (Figure B.1-52).The location was selected to protect the city of Puerto Ordaz from flooding.

Figure B.1- 52 Macagua Project. At the back the city of Puerto Ordaz. (S.Contreras)

Spillway Design

Macagua spillway design was mostly done by using large physical models. Best location of the spillway was selected in the area immediately upstream of the La Llovizna Falls. Discharge capacity is 30,000 m3/s following the PMF received from upper reaches of the Caroni River controlled by the discharges at Guri Dam, located 100 km upstream of Macagua Dam. The Spillway consists of an ogee with 15m of maximum head corresponding to PMF (Figure B.1-53 to B.1-56). It is a low ogee because the approach channel is only 8m below the ogee crest. Each of 12 bays is equipped with a radial gate operated by hydraulic cylinders. The ogee flows in a very short reach of chute and then a flat horizontal slab takes the flow direct to the river bed. Some 800 down-stream there is an average 12m fall i.e. the La Llovizna Falls, a local tourist attraction. On top of the spillway piers there is a national road provided with 8 lanes. Every two bays, there is a long wall that supports the road structure. The end of the road coincides with the end of the concrete slab and immediately after this joint the river bed was cleared horizontally to discharge the spill-way flows.

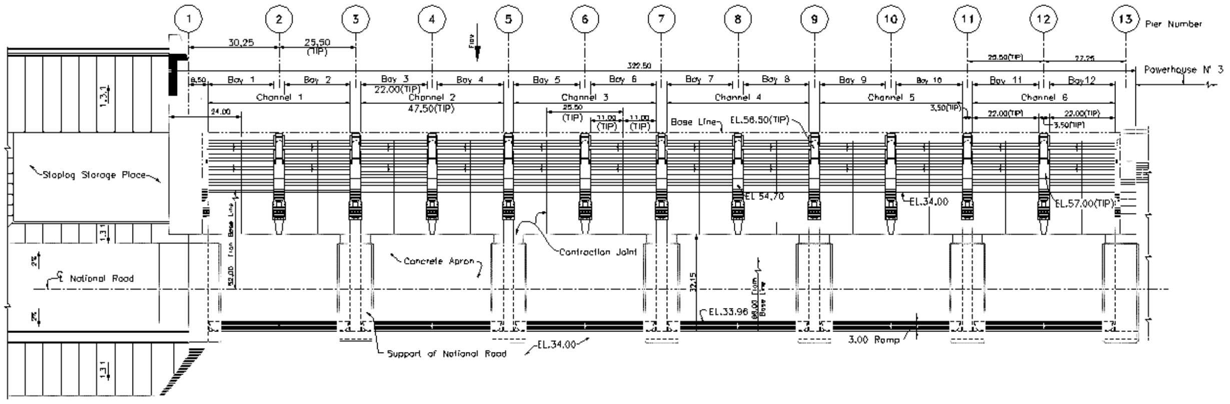


Figure B.1- 53 Macagua Spillway –Plan view (EDELCA)

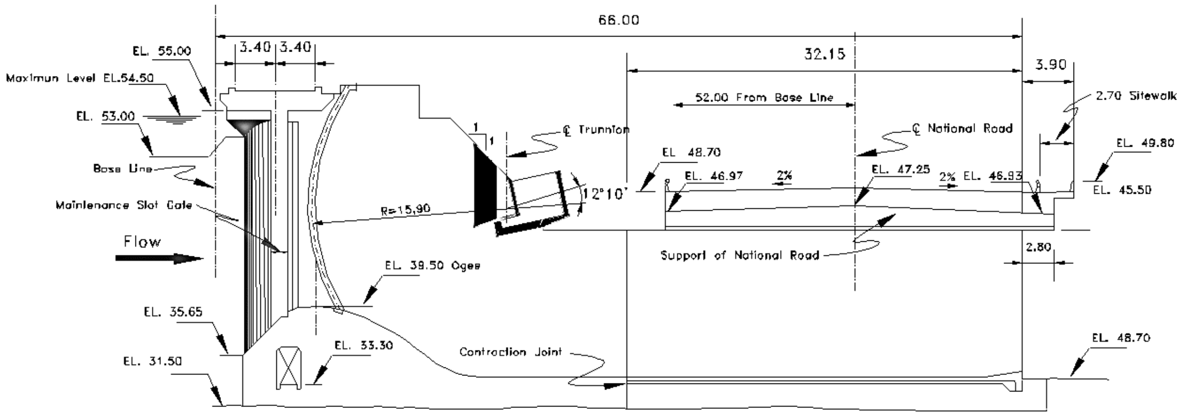


Figure B.1- 54 Macagua Spillway –Cross Section (EDELCA)

Figure B.1- 55 View of Macagua Spillway. (La Llovizna Falls are seen d/s of spillway).

Figure B.1- 56 Spillway and 8 lane national road. (EDELCA)

Operation of the spillway

Flow concentration in the spillway produced significant erosion in the nearby river bed, immediately downstream of the spillway (Figure B.1-57). At the very beginning of the project in 1997 a decision was taken to extend the concrete slab 40 m downstream to protect the spillway and national road.

Figure B.1- 57 Macagua Spillway rehabilitation works. Cleaning geologically weak areas in front of the spillway. (EDELCA)

Rehabilitation measures

Repairs were done by anchors (11 number bars) and concrete. However, as the scour was very close to the concrete slab that supports the road, an engineering decision was taken to extend the concrete slab downstream for a distance of 40 m from the end of the road (Figure B.1-59). Monitoring of the scour has revealed that it has stabilized showing scour holes not larger than 8m in depth (Figure B.1-59 and Figure B.1-60).

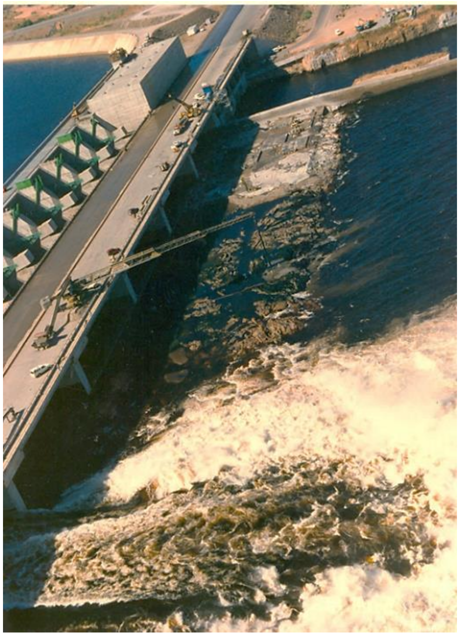


Figure B.1- 58 Macagua Spillway operating with concrete slab extended. (EDELCA)

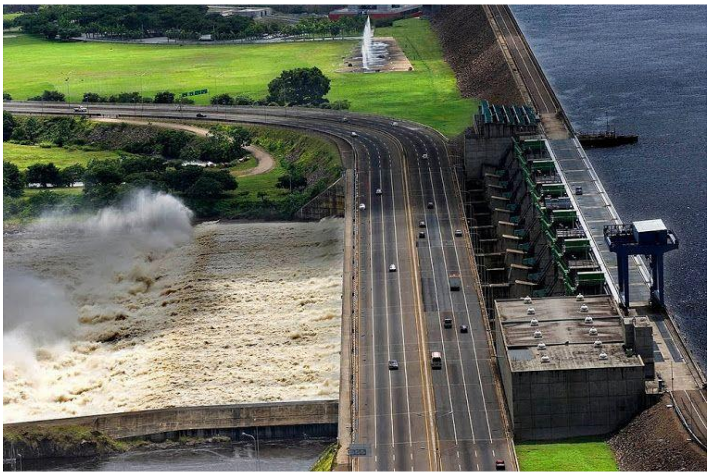


Figure B.1- 59 Macagua Spillway operating (Bolivar, M.)

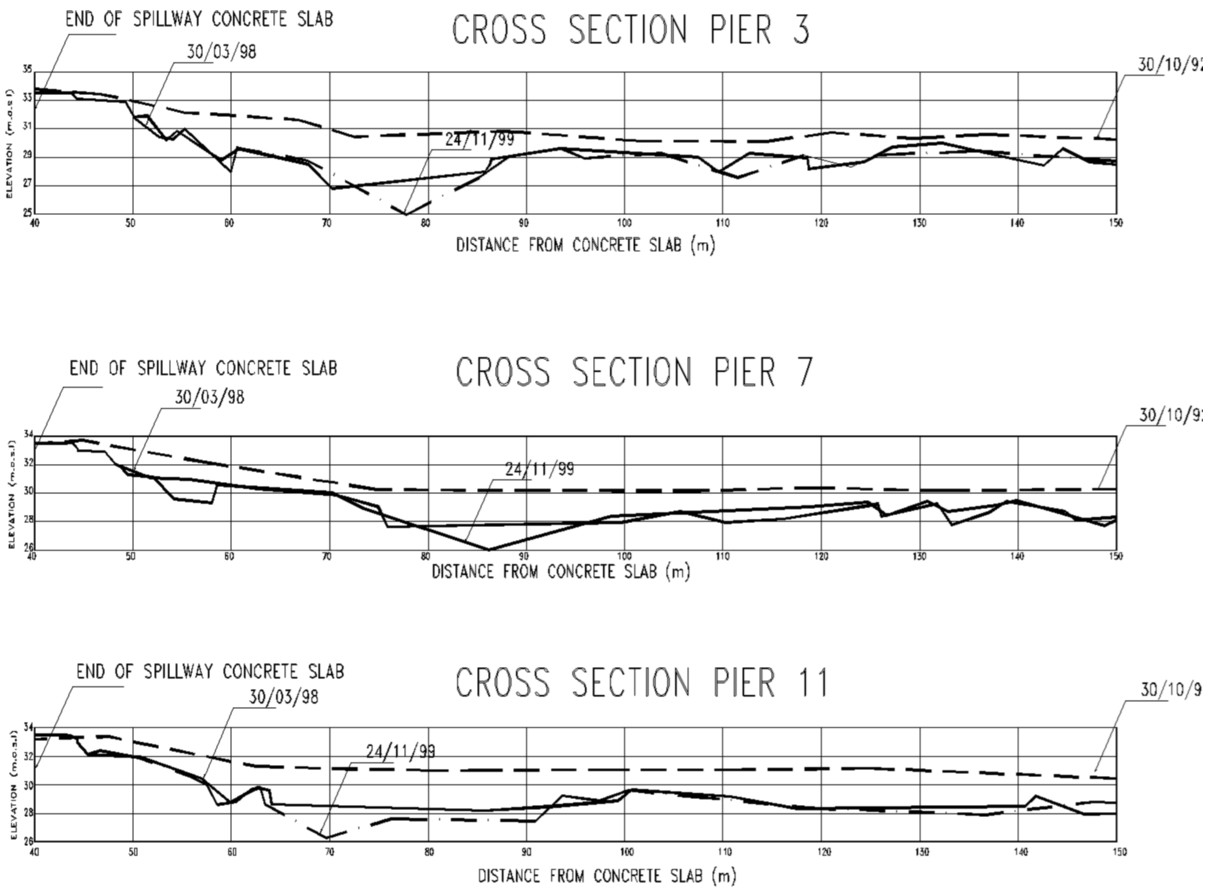


Figure B.1- 60 Surroundings downstream of Macagua spillway showing a maximum erosion of 6.00m (EDELCA)

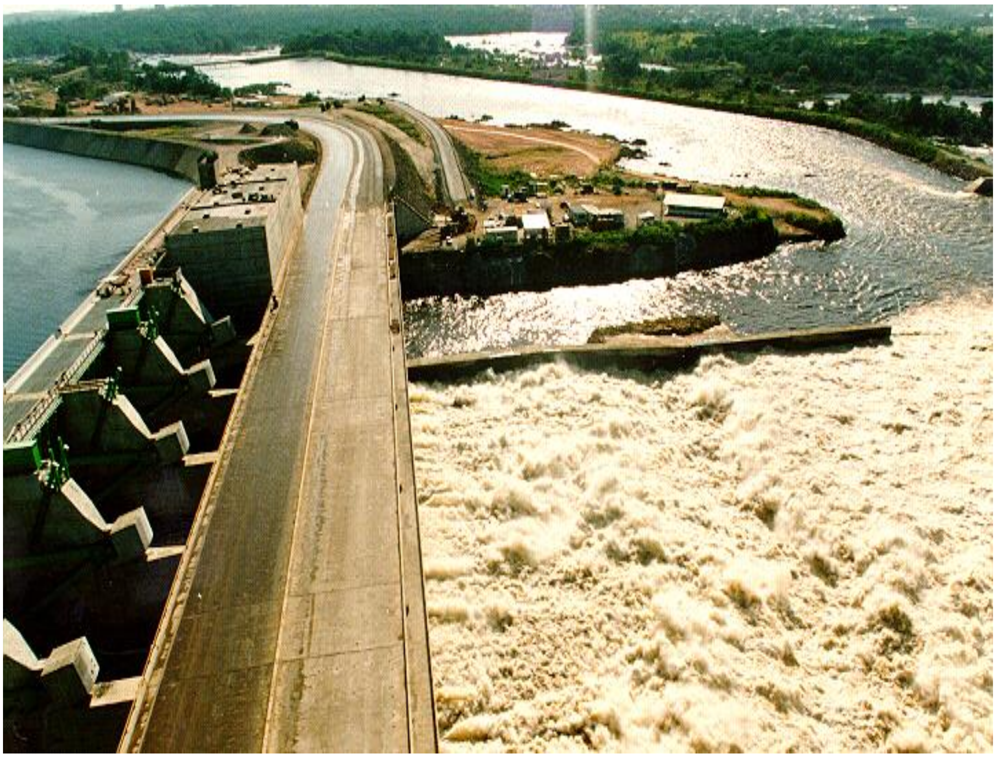


Figure B.1- 61 Macagua spillway operating with the extended concrete slab for 14,000 m^3/s, 45% of the design spillway flow. (EDELCA)

Damage to the banks of La Llovizna Park

During the river diversion stage, a flood with a peak of 9,000 m^3/s at the Macagua Spillway got concentrated in a few areas for the first time for more than 3 months. Damages occurred at the banks and pathways of La Llovizna Park (Figure B.1- 62). Banks were affected by intense wave action and up rush water movement that eventually eroded all the schist that outcropped at the banks. La Llovizna Falls were dis-figured because of the scour which took place on the fall over-flow edge (Figure B.1-63). Flow was concentrated in only a few sites, leaving large areas of the falls without water.

Figure B.1- 62 Flow distribution at the fall. (Marcano)

Figure B.1- 63 Extensive scour in the banks of La Llovizna Park. (EDELCA)

Rehabilitation works

Rehabilitation measures were taken in order to restitute the water distribution along the La Llovizna weir and, protection of the banks of La Llovizna Park. Water was diverted from the site and in 3 months flow distribution conditions were physically model tested.

Solution was found for both problems. An array of small dykes, 9 dykes of variable height, from 1.5 to 6m in height were proposed to be built at the weir section at different regions of the falls. The dykes were built with large rock drilled and anchored to the river bed to close all depressions and produce the prototype distribution expected. Two physical models were built, a sectional model scale 1:10 used to test the array of rock to be reproduced in the prototype and a 3-D Comprehensive physical model in which the best location for the 9 dykes was investigated (Figure B.1-64 and B.1-65). For the right bank, a protection of rock, anchored to sound rock, was built along all along the face of the bank (Figure B.1-65). Some excavation of the right bank was needed to improve water capacity of the channel.

Figure B.1- 64 Partial model of the dyke to study the cross section and the visual aspect of the array rock concrete. (EDELCA)

Figure B.1- 65 Physical Model of Macagua project. View of the model to study the dykes location. (EDELCA)

The system of 9 dykes was built and, flow distribution similar to that before the flood was achieved (Figure B.1-66 to B.1-70).

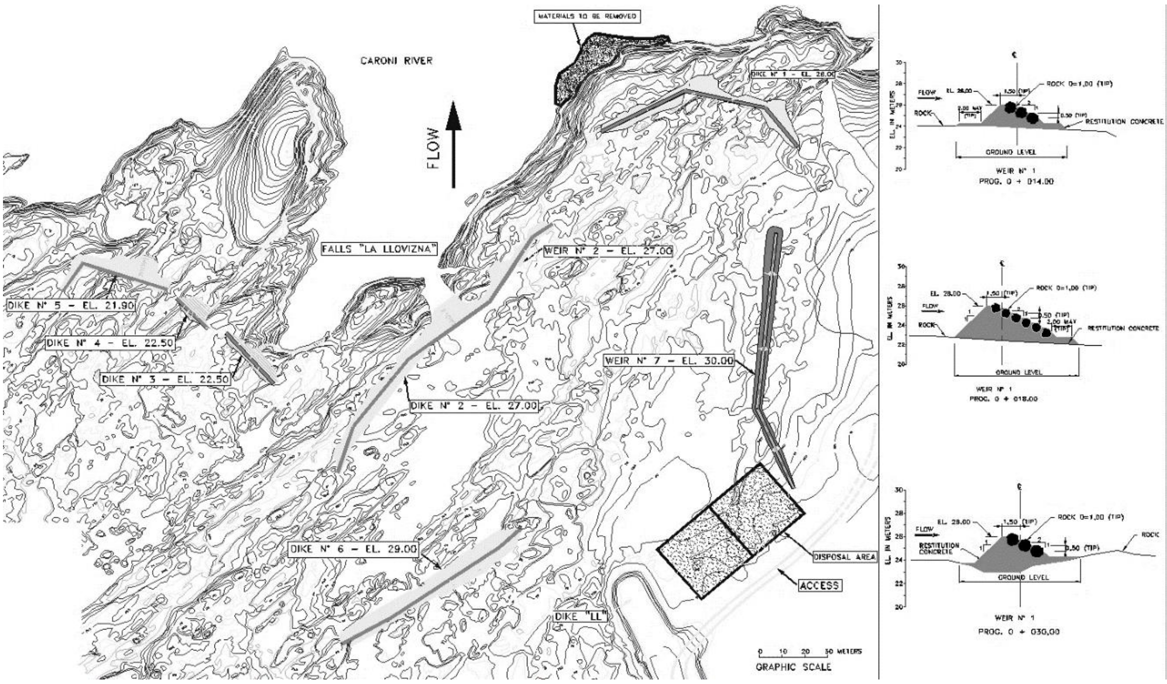


Figure B.1- 66 System of Dykes built to distribute the flow along the falls overflow weirs. (EDELCA)

Figure B.1- 67 Rehabilitation works to restore the flow distribution to La Llovizna Falls. (EDELCA)

Operation of the system of weirs and bank protection

After the flood, the park was closed for 5 months, the works were terminated and the flow was restituted to the fall. Flows have occurred particularly the last two years, 2008 and 2017 and flow over the dyke has been as much as 14,000 m^3/s, with depts. On the dyes estimated in 7m. Performance of the bank protection has also proved very effective.

Figure B.1- 68 The Falls operating with the Dykes system operating for ecological flows. (Routes.global)

Figure B.1- 69 Scale model 1:40 of the dyke system in the Macagua 3D physical Model. (EDELCA)

Figure B.1- 70 Macagua Spillway and flow behavior with Dyke System in operation. (Vilchez)

Caruachi Hydroelectric project

Caruachi Hydroelectric Project (Figure B.1- 71) is part of the Hydroelectric Development of Lower Caroni, located in South-east Venezuela and has been in operation for 17 years. It consists of a CFRD dam of maximum height 60 m and 800m in length, a 1,630 m long embankment dam, a spillway with capacity of 30,000 m^3/s equipped with 9 gates of 15.24m width x 21m height each and a Powerhouse equipped with 12 Kaplan Units of 216 MW each, totaling to 2,196 MW.

Figure B.1- 71 2,196 MW Caruachi Hydroelectric Project in the Lower Caroni, Venezuela (EDELCA)

The Spillway

The 45m long spillway of Caruachi Dam was designed to fulfill two purposes: to pass the diversion flood and to serve as a service spillway for the useful life of the project. Both functions were successfully accomplished by designing a double body spillway (Figure B.1-72 and B.1-73). The lower (temporary diversion) and the upper body the permanent service spillway, were designed for passing 14,000 and 30,000 m^3/s discharge, respectively. The specific flow of the service spill-way was 178 $m^3/s/m$. The river bed consisted of competent granitic Gneiss belonging to Roraima geologic formation as part of the ancient Guayana Shield. The head is only 35m from maxi-mum reservoir elevation to bucket lip elevation. The jet is thrown very close to the toe of the bucket into a 15m deep pre excavated plunge pool. Spillway operation rule envisages starting the spilling first from the central 3 bay channel, then from the left 3 bay channel, followed by the right 3 bay channels. This type of operation was adopted to protect the right abutment structures, thus releasing flows more frequently by the central and left channels.

Figure B.1- 72 30,000 m³/s Caruachi Spillway front view. (EDELCA)

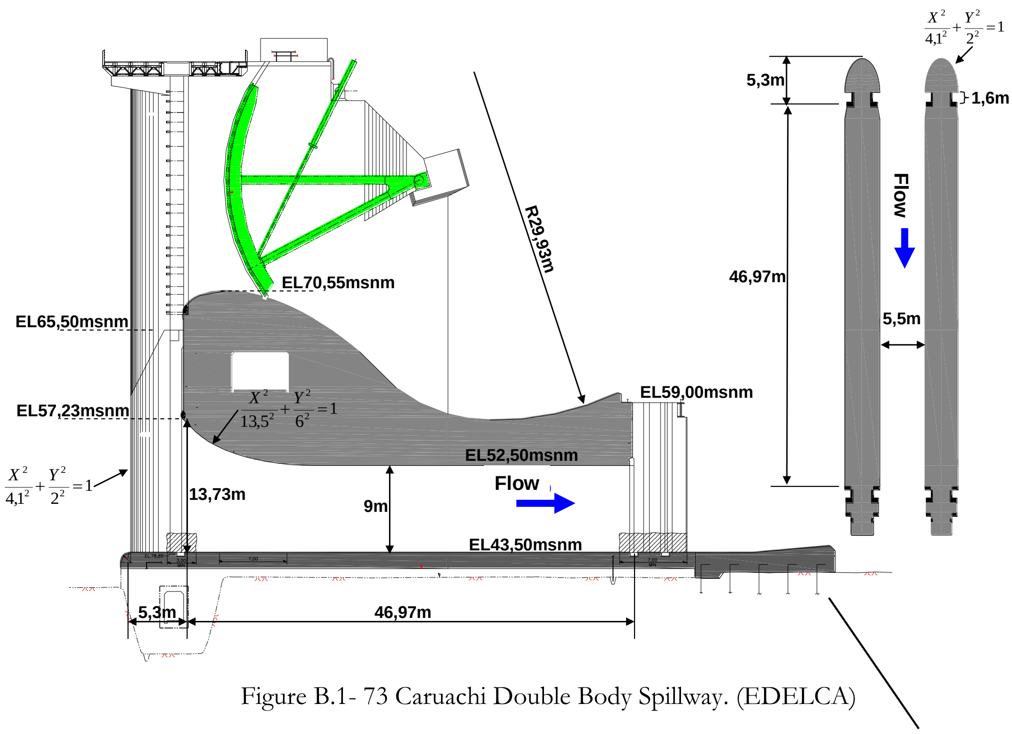


Figure B.1- 73 Caruachi Double Body Spillway. (EDELCA)

The Lower Body

The lower body consisting of 18 bottom sluices, 9m high and 5.5m wide, were designed to operate under submerged conditions to pass the 14,000 m^3/s diversion flood Figure B.1-74 shows photo during construction). At the end of the First River Diversion Stage, these 18 sluices were closed by means of a 50 Ton service gate located downstream (Figure B.1-75 first photo). A second slot was located at the upstream end of the structure, just in case of jamming of the downstream service gate as a second contingency that could be covered by a 70 Ton emergency gate, designed to stop a sluice flow of 1,100 m^3/s (Figure B.1-75 second photo). The lower body or diversion body finishes with a concrete horizontal slab provided with a small ramp, designed to direct the diversion flow into the pre excavated plunge pool for all flows from the average of

5,000 m^3/s to the maximum flow of 14,000 m^3/s. Last sluices were closed at a flow of 1,100 m^3/s each and a head difference of 25m.

Figure B.1- 74 Caruachi Hydroelectric Project - Double Body Spillway during construction; 18 no.,9m x 5.5m sluices (lower body, for river diversion) and 9 gates (15.24x21m) for service spillway (EDELCA)

Figure B.1- 75 Caruachi Hydroelectric Project-River diverted through 18 no. (9m x 5.5m) bot-tom sluices. (EDELCA)

The upper body

The upper body or service spillway consists of 3 channels of 3 bays each, equipped with radial gates (Figure B.1-76). Each bay is separated by 4m width walls. Upper body service spillway lay out is very conventional. Crest of the spillway follows a Creager profile, followed by a short chute and finishing in a flip bucket, 30° exit angle and 25m radius that throws the flow jet to the pre-excavated plunge pool.

Figure B.1- 76 Caruachi Hydroelectric Project- Surface spillway operating. Note jet being thrown very close to the structure and apparent good energy dissipation downstream of the impact zone. (EDELCA)

The energy Dissipater

Caruachi Energy dissipater consists in a flip bucket, 30 m radius and an exit angle of 30°. Jets are thrown to a pre excavated plunge pool 25m deep and 180m in length. Pre excavated plunge pool was selected from investigations in a comprehensive Physical model of scale 1:80 built in EDELCA's Hydraulics Laboratory. Caruachi spillway has operated for 17 years showing excellent performance as it was expected in the design stage. Energy dissipation is efficiently achieved much closed to the plunge pool. Not significant evidence of additional erosion has been observed in the channel including rip rap placed close to the spillway (Figure B.1- 77 to B.1-81).

Figure B.1- 77 Caruachi Hydroelectric Project –Spillway pre excavated plunge pool. (EDELCA)

Figure B.1- 78 Left and right sides of the pre excavated plunge pool. (EDELCA)

Figure B.1- 79 Short concrete slab anchored to the rock to receive the flow from the sluice at the diversion stage. Flow from the permanent spillway falls from the flip bucket into the plunge pool constructed in granitic gneiss. (EDELCA)

Figure B.1- 80 Caruachi Hydroelectric project. Plunge pool during first filling. (EDELCA)

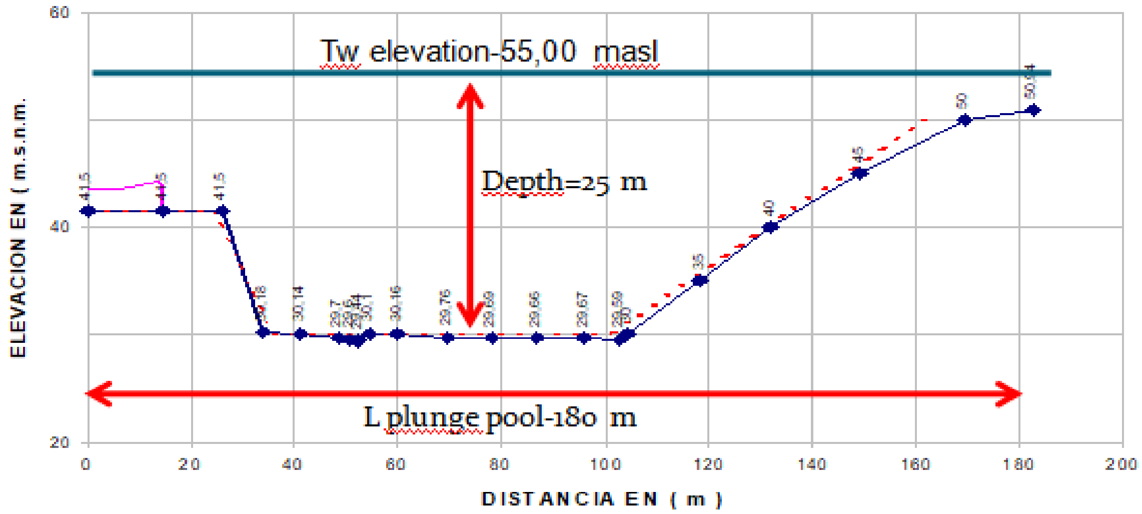


Figure B.1- 81 Caruachi Spillway - Pre excavated plunge pool (Marcano, 2002)

Modeling Investigations

A number of Physical and numerical models were used to study the different problems related to the design and performance of the double body spillway (Marcano et alia, 2004). These investigations included the operation of a 3D Comprehensive 1:80 Scale Model, a Sectional 2-D Double body Spillway Physical Model 1:40 Scale (Marcano et alia, 2002) and a 1:13 Scale Physical Mod-el to study the Emergency Closure (Marcano et alia, 2002) (See Figure B.1-82 to B.1-85 for the physical models). Numerical Models 3D have been also used to evaluate different problems of the project (Montilla et alia, 2005, Marcano et alia, 2004). See (Figure B.1-86 and B.1-87)

Figure B.1- 82 Caruachi Hydroelectric Project –Comprehensive 3D Physical Model Scale 1.80. Frontal view. (Marcano, 2002)

Figure B.1- 83 Caruachi Hydroelectric Project –Comprehensive 3D Physical Model Scale 1.80. View of double body spillway from upstream. (Marcano, 2002)

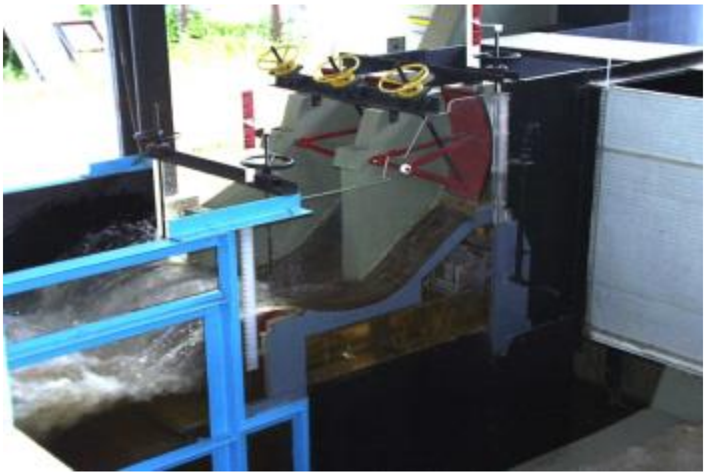



Figure B.1- 84 Caruachi Hydroelectric Project- Surface service spillway with flip bucket and pre excavated plunge pool. (Marcano, 2002)


Figure B.1- 85 Caruachi Hydroelectric Project. Sluice and gate model Esc 1:13 to study down pull forces during Sluice Emergency Closure. (Marcano, 2002)

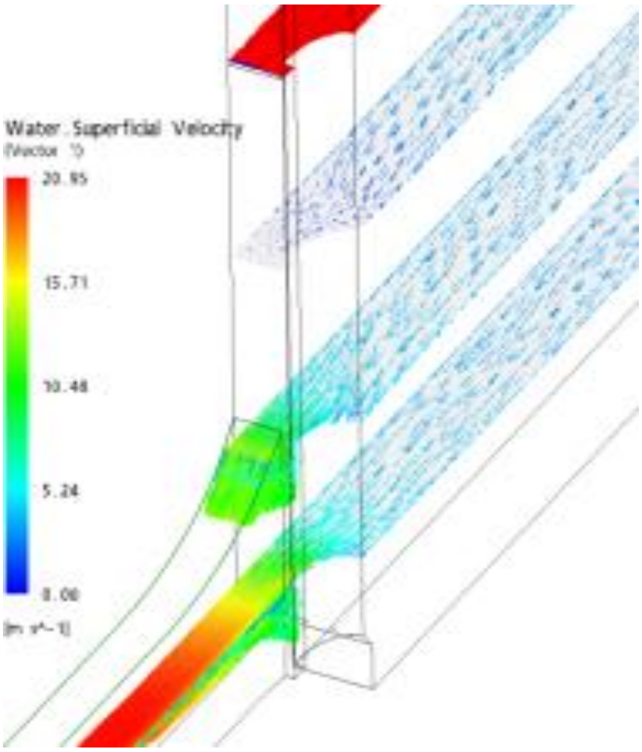


Figure B.1- 86 Caruachi Hydroelectric Project CFX 3D Simulation of Flow at the Sluice Intake (Montilla et alia, 2009)

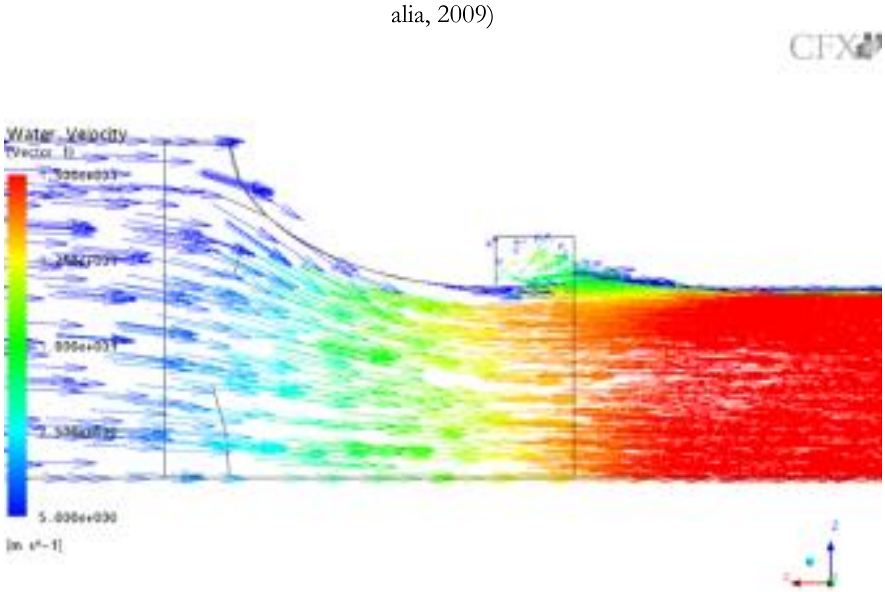


Figure B.1- 87 Caruachi Spillway-Lower Body 3D Plan simulation (Montilla et alia, 2009)

Tocoma Hydroelectric Project

Tocoma Hydroelectric Project, which is the last project of the Hydroelectric Development of Lower Caroni, located in South-east Venezuela is presently under construction. It consists of a CFRD dam of maximum height 35 m and 5400m in length, a 800 m long embankment homogenous dam, a spillway with capacity of 29,840 m^3/s, equipped with 9 gates 15.24m width x 21m height each and a Powerhouse equipped with 10 Kaplan Units of 220 MW each totaling to 2200 MW, (Figure B.1-88).

Double Body Spillway

Similarly to Caruachi Hydroelectric Project, Tocoma spillway consisted in a double body structure, the upper body for the permanent spillway included 9 bays, 15.24 m wide and 22 m height.

Pre excavated Plunge pool

A pre-excavated pool was designed as indicated by mobile bed tests in a 3D comprehensive Physical Model. However, at the spillway site in Tocoma, after the excavation works a band of weathered material was found crossing the area in front of the spillway , just where the jets falls into the pool (Figure B.1-88) . In was considered prudent to take some measures to protect the spillway and the river channel.

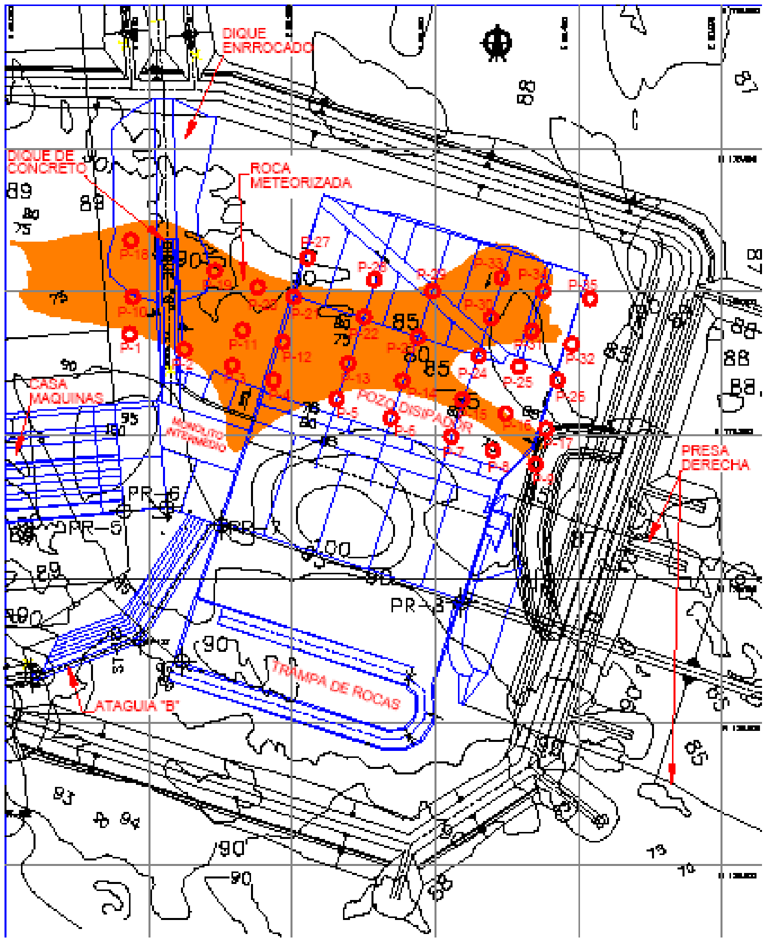


Figure B.1- 88 Tocoma Hydroelectric Project. Spillway location and weathered rock shown by orange color. (Courtesy of CORPOELEC)

For that reason, two important measures were taken to protect the spillway against potential erosion and the downstream channel for rock elements coming from the spillway plunge pool which could eventually be transported into the channel and produce rising of the tail water levels and subsequently, energy loses. First of all the Tocoma Spillway was provided with a anchored concrete slab, in the downstream slope of the plunge pool. Additional protection against possible erosion and transport of material to the tailrace was provided by a 150 m wall as indicated by the investigations carried out in a 1:80 scale Tri-dimensional Physical Model (Figure B1-89). However, because of continuous operation of the spillway for the last 5 years, it has not been possible to monitor the plunge pool bed; so the effect of the above cited modifications is not known.

Figure B.1- 89 lateral view of concrete slab on the upstream slope of Tocoma Pre excavated Plunge pool and a wall built on the left bank of the spillway to separate flows from the Powerhouse from flows from the spillway. (CORPOELEC)

B2-Taum Sauk Upper Reservoir Dam Failure - December 14, 2005

Abstract

The upper reservoir of the Taum Sauk pumped storage hydroelectric plant which is located near St. Louis, Missouri failed in December 2005. Taum Sauk is a reversible pumped-storage project having a 450 MW, two- unit pump-turbine. The upper reservoir structure which is located atop Proffit Mountain is 800 feet (244 m) above the hydroelectric plant and was designed and built in the early 1960s as a concrete-faced rockfill dam (CFRD) with a kidney-shaped dike (Figure B.2-1). On December 14, 2005, the water level sensing and control system of the reservoir including a backup set of instruments failed to function properly and caused over pumping of water into the reservoir and breached a 700 feet (213.4 meter) section of the dam surrounding the reservoir. The breach released more than a billion-gallon torrent of water that swept away at least two homes and several vehicles and critically injured three children. A post failure investigation was conducted to review the reservoir design and operational procedure. The project's emergency action plan and its execution during the incident were also reviewed.

Taum Sauk Dam Overall View of the Breached Dike (Damfailures.org)

Figure B.2- 1 Kidney shaped Taum Sauk Dam (Missouri) (Damfailures.org)

In the following sections, a summary of this dam failure incident is provided as outlined below.

- The Taum Sauk dam failure incident
- Taum Sauk hydroelectric project
- Taum Sauk Plant Instrumentation and Controls
- The post failure investigation
- Standard Operational Procedures
- Upper Reservoir Dike Construction versus Design
- Upper Reservoir Dike Design and Construction Practices (1963 versus 2006)
- Emergency Action Plan:
    - Notification Flow Charts
    - Detection of Emergencies
    - Inundation Maps - Upper Reservoir
    - Training and Exercises
- Owner's Account of the December 14, 2005 EAP Activation and Coordination
- Remarks:
    - Use of sophisticated water-level sensors, instrumentation, and control system requires periodic inspection, calibration, verification, and training etc. especially when involving remote operation.
    - A modern instrumentation and control system can be supplemented by conventional devices which are simpler to understand, maintain, and use.

- Installation of monitoring camera may be useful for added safety precaution.
- Modern electronic instrumentation and control systems require serious specialist-level care and maintenance.

The Taum Sauk dam failure incident

The Taum Sauk pumped storage hydroelectric plant is located in the St. Francois mountain region of the Missouri Ozarks approximately 90 miles (145 km) south of St. Louis near Lesterville, Missouri. The plant is located off channel to East Fork of the Black River and is operated by the AmerenUE electric company. The Taum Sauk is a reversible pumped-storage project having a 450 MW, two- unit pump-turbine. The upper reservoir structure which is located atop Proffit Mountain is 800 feet (244 m) above the hydroelectric plant. It was designed and built in the early 1960s as a concrete-faced rockfill dam (CFRD).

On Wednesday, December 14, 2005, the water level sensing and control system of the upper reservoir failed to function properly that caused over pumping of water into the reservoir and breached a 700 feet (213.4 meter) long section of the dam surrounding the reservoir by overtop-ping. The breach resulted in the reservoir emptying its water down Proffit Mountain. The Associated Press reported that the retaining wall around the huge mountaintop reservoir in the Ozarks collapsed before daybreak, releasing more than a billion-gallon torrent of water that swept away at least two homes and several vehicles and critically injured three children.

Just after 5 a.m. a V-shaped, approximately 600-foot breach opened up at the mountain top reservoir and in a matter of minutes the 50-acre reservoir had emptied itself out with terrifying effect, turning the surrounding area into a landscape of flattened trees and clay-covered grass.

AmerenUE chairman and chief executive Gary Rainwater said that the plant's automated instruments had allowed pumping too much water into the reservoir and caused it to rupture. Un-fortunately, a backup set of instruments failed to recognize the problem and took no-corrective actions.

The office of energy projects at the Federal Energy Regulatory Commission (FERC), said the plant, including the reservoir, was inspected in August (2005), only about 4 months before the failure and found to be properly operated and maintained. An exhibit providing a brief description of the Taum Sauk upper dam failure event is shown in Figures B.2-2 and B.2-3.

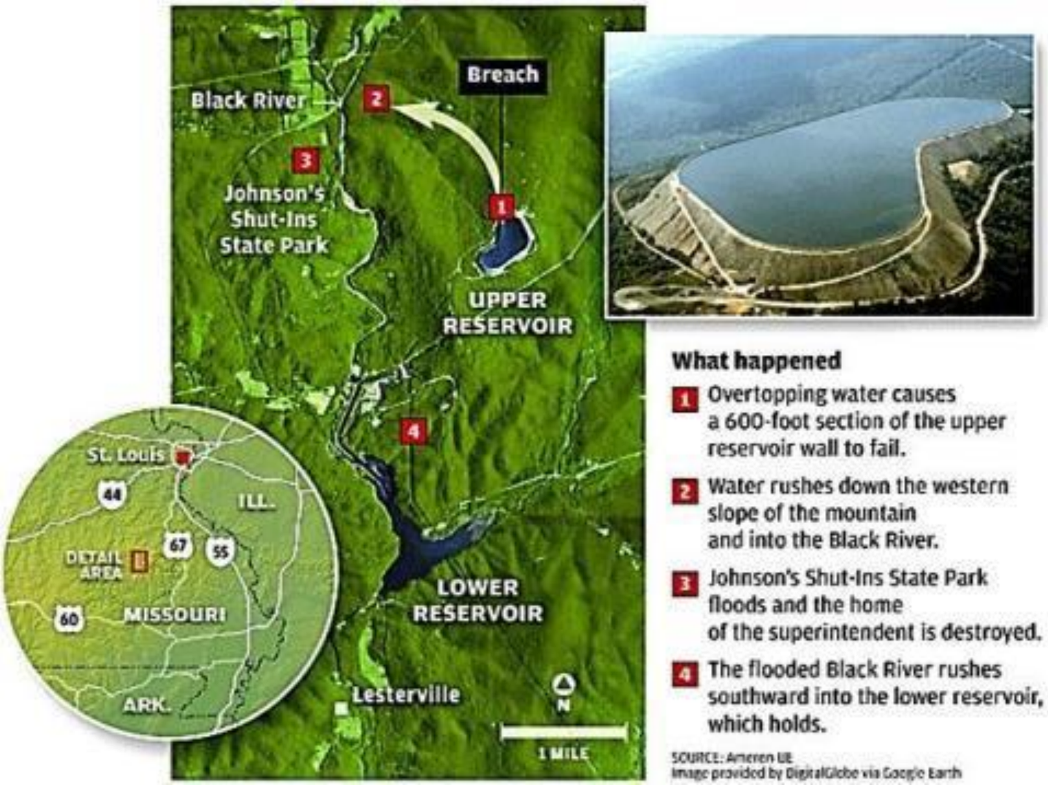


Figure B.2- 2 A brief description of the breach incident. (AmerenUE).

Figure B.2- 3 A large section of the upper reservoir failed, draining over a billion gallons of water in less than half an hour (left) and the flood path downstream of the breach section (right).

Taum Sauk hydroelectric project

The Taum Sauk Plant is located in Reynolds County, Missouri, on the East Fork of the Black River, approximately 90 miles southwest of St. Louis, Missouri. It has an upper reservoir that is impounded by a rockfill dike and a Lower Reservoir that is impounded by a dam across the East Fork of the Black River. The two original reversible pump-turbine units were each capable of generating 175 megawatts of power. They were upgraded in 1999 to units capable of 225 megawatts each.

The original Upper Reservoir structure designed and built in the early 1960s was a concrete-faced rockfill dam (CFRD). It is a reversible pumped storage project used to supplement the generation and transmission facilities of AmerenUE, and consists basically of a ridge top upper reservoir, a shaft and tunnel conduit, a 450-MW; two-unit pump-turbine, motor- generator plant, and a Lower Reservoir (see Figures B.2-4 to B.2-6).

The Upper Reservoir (A) was breached "at the northwest corner" (B). The hydroelectric plant is located at (C). Several people were rescued from Johnson Shut-Ins State Park (D).

Figure B.2- 4 This topographic map shows the Taum Sauk Hydroelectric Plant complex.

Figure B.2- 5 A view of upper reservoir (left) and powerhouse in relation to reservoir (right).

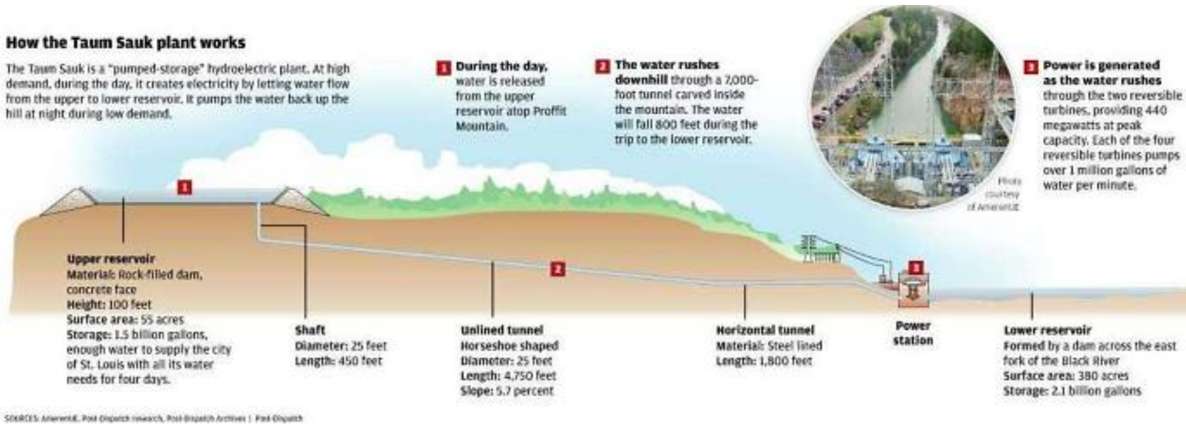


Figure B.2- 6 A brief pictorial description of how the Taum Sauk works (AmerenUE)

The upper dam is a continuous hilltop dike 6,562-ft-long forming a kidney-shaped reservoir (see Figure B.2-7). The dike is a concrete-faced dumped rockfill dam from the foundation level to elevation 1,570.0 ft and a rolled rockfill between Elevation 1,570 and 1,589 (see Figure B.2-8). A 10-foot-high, l- foot-thick reinforced concrete parapet wall at the top of the fill extended the crest to elevation 1,599 ft at the time of the original construction. Since construction, settlement of the rockfill varied between 1 and 2 ft with the lowest area found after the breach at panel 72. At panel 72, the top of the embankment was at elevation 1,586.99 ft and top of the parapet wall was at elevation 1,596.99 ft.

The lower reservoir dam is located in a narrow steep-sided gorge just downstream of the junction of Taum Sauk Creek and the East Fork of the Black River. It forms the Lower Reservoir with a surface area of 395 acres and a water level at the spillway crest. The canyon at this location is in exposed hard blocky rhyolite rock of good quality. The Lower Reservoir design volume at the spillway crest is 6,350 acre-feet.

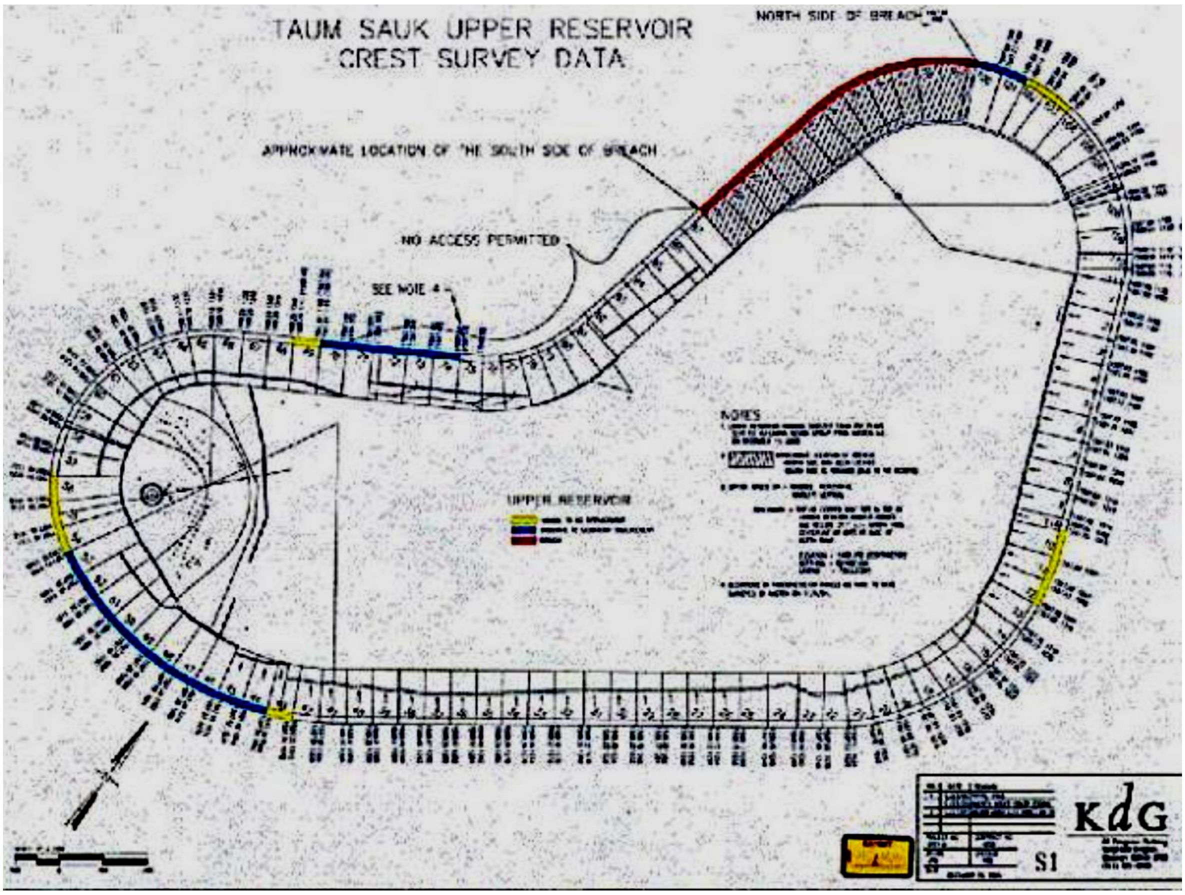


Figure B.2- 7 Taum Sauk upper reservoir dam, from KDG, Maryland Heights Missouri.

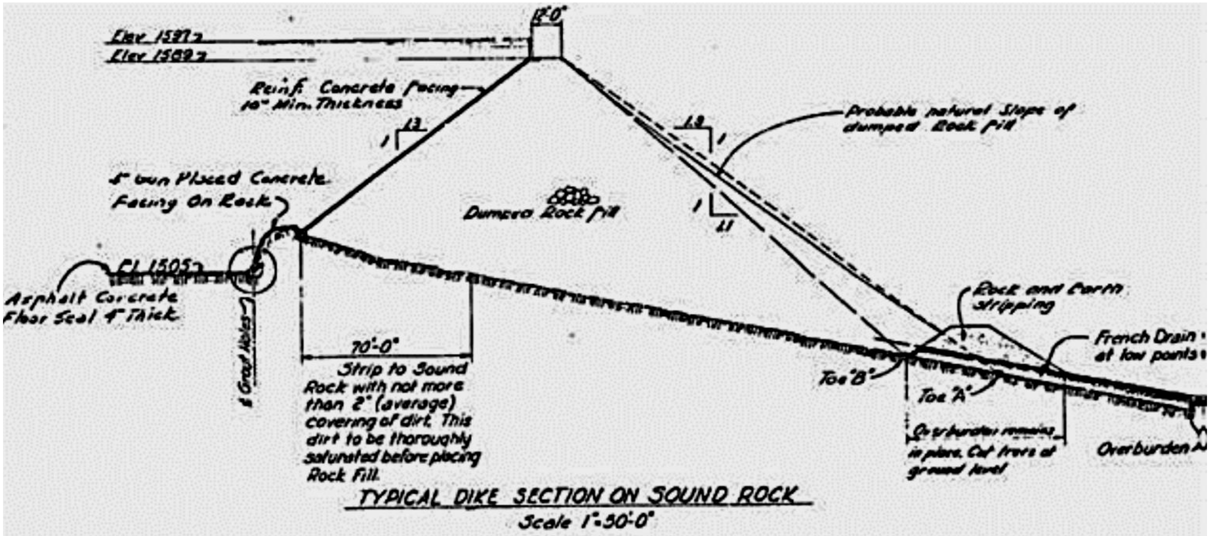


Figure B.2- 8 Taum Sauk upper dam cross section from original design drawings (FERC; 2006)

Figure B.2- 9 Reconstruction process of Taum Sauk dam with new design (gotops.com, 2010)

Taum Sauk Plant Instrumentation and Controls

Since the upper dam breach happened due to the malfunction of the plant's automated instruments and associated controls that allowed pumping of too much water into the reservoir and caused it to rupture, and the failure of the backup set of instruments to recognize the problem and to take corrective actions, it is important to understand the instrumentation and control system of the Taum Sauk plant as a lesson learned.

The project is controlled through a telephone and microwave system from the Osage Plant at the Lake of the Ozarks, under the direction of the load dispatcher in St. Louis. Both units can be put on full load in a few minutes.

As indicated by AmerenUE, at the time of the December 14, 2005 breach, the plant's automated instruments had allowed pumping too much water into the reservoir and caused it to rupture. A backup set of instruments failed to recognize the problem and took no corrective actions.

The upper reservoir control system consisted of two sets of sensors sending the signals through three independent PLC computers. One set of sensors were two Druck pressure transducers used to monitor reservoir levels (the third Druck pressure transducer was not used due to inaccurate readings).

The second set of sensors consisted of four Warrick Conductivity sensors. Two of the Warrick sensors (HIGH and HIGH-HIGH) were to determine if water levels in the upper reservoir were too high. The other two Warrick sensors were to determine if water levels in the upper reservoir were too low. Activating these sensors would start a hard shutdown of the generator/pump units.

The reservoir monitoring system (original) consisted of: (1) three Warrick conductivity sensors at elevations 1,501, 1,506 and 1,508, (2) a skate type system (i.e., a float riding on a cable guided roller assembly in a pipe) to monitor upper reservoir levels for normal shutdown of the units, and (3) a set of mercury switches tied to a float in a stilling well for High and High-High backup pump shutoff. There was an encoder and chart recorder on the skate system to provide level indication and recording. Components of the system were anchored to the concrete face of the dam.

In 1994, a differential pressure transducer was added to provide secondary level indication at the plant. In 2000, the original skate system, encoder, and chart recorder were replaced with a differential pressure level transducer, Programmable Logic Controller (PLC), and a digital level indicator at the upper reservoir.

All of the upper reservoir level control and protection devices were replaced when the geomembrane liner was installed at the end of 2004. Three General Electric Druck Model PTX 1230 100 psi piezo resistive micro machined silicon strain gauge pressure transducers (referred to as Duck pressure transducers or transmitters) were installed for normal shutdown of the units.

The Low and Low- Low Warrick conductivity sensors were replaced in kind. The High and High-mercury switches were replaced with Warrick conductivity sensors. The upper reservoir PLC was replaced with an Allen-Bradley PLC. The unit shutdown relays at the plant were replaced with Allen-Bradley PLCs. The level indicators, alarming, and data acquisition systems were replaced with a Wonder-Ware Operator Interface.

A structural design of the Upper Reservoir Control System (December 2004) is shown in Figure B.2-10 below while Figures B.2-11 and B.2-12 show photos of instrumentation cabinet and location, respectively.

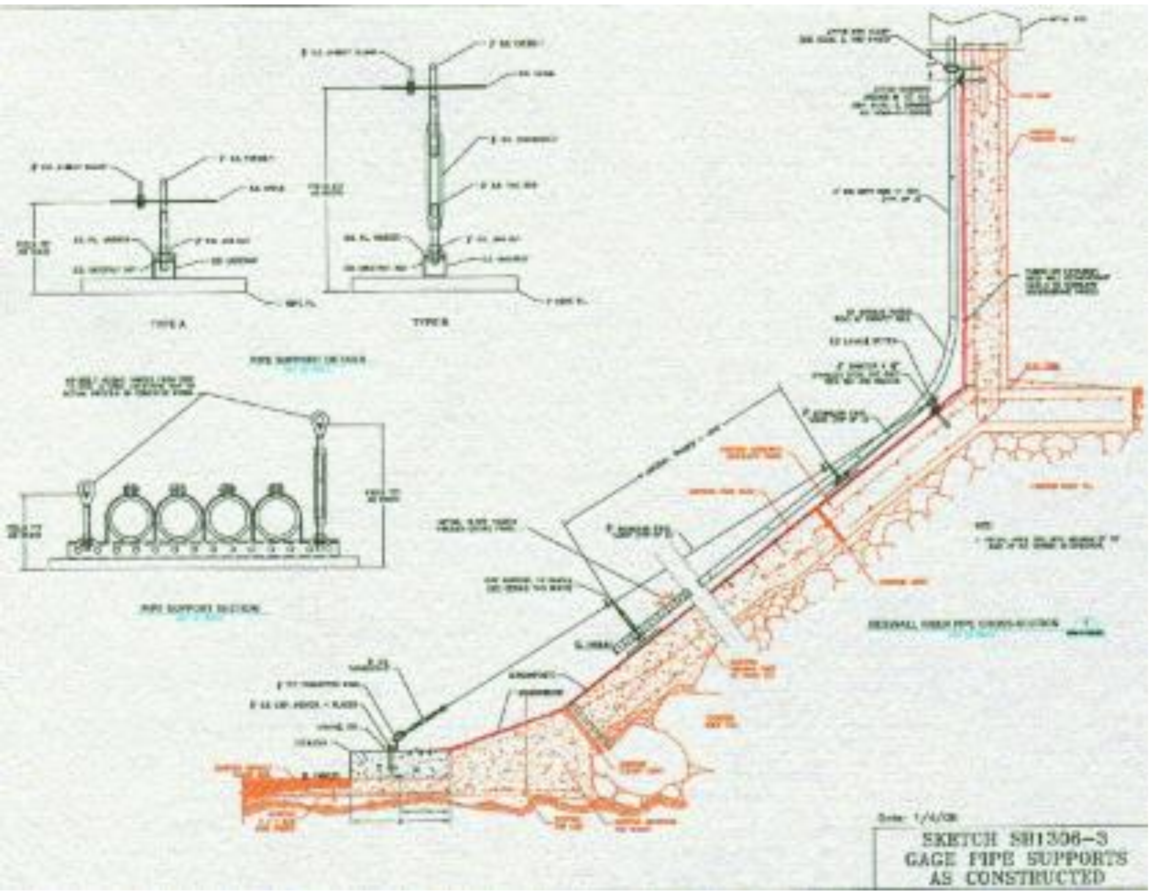


Figure B.2- 10 Design (December 2004) of the upper reservoir control system

Figure B.2- 11 Cabinet containing the Duck pressure transducer and Warrick conductivity sensor wiring (left) and Bowing of HDPE Tube position on December 15, 2005 (Courtesy of Missouri DNR)

Figure B.2- 12 Instrumentation location relative to the water conveyance shaft (left) and high-water marks on HDPE Tubes, (Courtesy of Missouri DNR)

A summary of the instrument data examination is provided as follows:

1. The variability in the system output when the reservoir was in pump mode appeared to be within the specification after initial installation.

2. Increases in variability of the Druck pressure transducer signals and output appear to increase from September 2005 through December 2005.

3. Variability appears to be greater than the sum of the electronic error and the rate of reservoir rise. Variability up to +/- 1.75 feet occurred December 13, 2005.

4. The greatest fluctuation in the output occurs during the pump cycle when the reservoir elevation is between 1545 ft and 1565 ft.

5. It is possible there is a relationship between the movement of the instrumentation pipes and the turbulence caused by the pumping cycle

The post failure investigation

A report prepared by Paul C. Rizzo, Associates (PCR) in 2006 on the Taum Sauk Plant upper dam breach and a news report by the Associated Press provide good insight on the incident.

As reported by the Associated Press, water released from the reservoir normally rushes down a 7,000-foot shaft and tunnel and spins the turbines to generate electricity. In Wednesday's accident, water gushed through the breached section and streamed down the side of the mountain and into a valley, draining the reservoir like a bathtub. The V-shaped breach (about 600-foot) opened up just after 5 a.m. At 5:12 a.m., the water level in the reservoir was high, according to AmerenUE, but by 5:24 a.m., it registered as low by the instrument. The water eventually flowed back into the Black River. Soon after the break, police and the National Weather Service urged the 150 residents of Lesterville to move to higher ground. But by midday, once the water had flowed back into the river, the danger had passed. Emergency workers said they saw two tractor- trailers pushed about 150 yards off the road, two pickup trucks and a car tossed into a field, and a house was totally gone. Figures B.2-13 to B.2-15 photographs of the breach section.

Figure B.2- 13 Weathered rock and discontinuous clay seam foundation just downstream of fish-pond area

Figure B.2- 14 Left side of breach (left) and right side of breach showing layering of embankment (right)

Figure B.2- 15 Erosion at Panels 48 and 49. Note scarps that may be the results of a localized slope failure. (FERC, 200&)

Damage judged as significant. Estimate 0.5 ft of overtopping (left). Scarp near toe of parapet wall footing. Note erosion rut adjacent to the footing.

The pneumatically placed upstream concrete face slab has a design thickness of 10 inches, and is reinforced with No. 7 bars at 12 inches, both ways. In actual placement, the slab thickness averaged nearly 18 inches due to the unevenness of the rockfill. There are no horizontal joints except at the junctures with the Parapet Wall and the foundation cutoff-slab. The face slab was placed in panels, 60 feet wide at their widest dimension. Expansion joints between the slabs, to accommodate movement caused by settlement of the rockfill, used asphaltic expansion joint material and U-shaped copper water stops.

A reinforced concrete plinth was provided at the toe of the concrete face. Where the natural rock surface was substantially higher than the reservoir floor, the rock was excavated on a near vertical slope and the plinth was placed at the top of the excavated rock. In these areas, the rock cut between the reservoir floor and the plinth was sealed with a 4-inch layer of wire mesh reinforced shotcrete. The entire reservoir bottom was sealed with two 2-inch layers of hot-mix asphaltic concrete placed over leveled and compacted quarry muck. Around the edge of the asphaltic concrete, a single line grout curtain was constructed to limit seepage under the Dike.

The Upper Reservoir was constructed without a spillway. It was reasoned (Ref. 4) that it has no drainage area and the only incoming flow is by pumping and direct rainfall. A system of redundant water level instruments was designed to prevent overtopping of the Dike by pumping. These instruments will be addressed later.

The Taum Sauk Powerhouse is located at the upstream end of the Lower Reservoir about two-miles from the Upper Reservoir as illustrated on Figures B.2- 4 and B.2- 5. It is situated in a deep, narrow canyon through which a tailrace channel was excavated to connect to the East Fork of the Black River as shown on Figure B.2- 5. The Powerhouse is connected to the Upper Reservoir via a concrete and steel-lined shaft and tunnel. The initial reversible pump-turbine rating for each unit was 175 MW, but these have been upgraded such that the total plant capacity is now in the range of 450 MW. The tailrace that leads to the Lower Reservoir is about 65-feet wide and 2,000-feet long.

The Upper Reservoir Outlet is the power conduit that consists of a 451-foot deep, 27.2-foot diameter vertical-shaft shaped at the top as a typical "morning glory." The top 110 feet of the shaft is concrete-lined. It connects to a 4,765-foot long, 25-foot diameter, unlined horseshoe tunnel sloping at 5.7 percent which ties to a horizontal 1,807-foot long, 18.5-foot diameter steel-lined tunnel and a short penstock that bifurcates to the pump-generating plant. The morning glory intake is located in the southwestern portion of the Reservoir in a localized area of the floor that is 20 feet lower than the rest of the Reservoir floor to suppress vortex development. Two 9- foot ID spherical valves in the Powerhouse control flow from the Upper Reservoir.

Standard Operational Procedures

The Taum Sauk Plant is a peaking and emergency reserve facility. A typical daily cycle in the summer is to generate from about 2 PM until 7 PM by releasing water from the Upper Reservoir through the powerhouse turbines to the Lower Reservoir and pump from the Lower Reservoir to the Upper Reservoir from 11 PM until about 6 or 7 AM. Generation and pump start as well as the respective durations are determined by system needs and controlled from the power company, AmerenUE's Osage Plant. In the winter, the number of cycles is typically less, with no cycles on some days. Operation in the fall and spring typically follows ambient temperature and system load requirements.

The normal maximum level for the Upper Reservoir is El. 1,596. The Upper Reservoir can be drawn down to El. 1,525 feet and possibly to El. 1,515 if only one turbine/generator unit is operating without causing problems with the hydro machinery. The normal minimum level in the Lower Reservoir is El. 736. Although this is above the bottom of the Lower Reservoir, operation below this level draws debris up the Powerhouse tailrace channel. The debris interferes with the pumping operations and sets the practical minimum water level elevation. The normal maximum water level is El. 749.5 feet or 6 inches below the spillway crest. AmerenUE operates the gates manually such that outflow over the spillway and through the sluice gates is equal to Lower Reservoir inflow to satisfy Federal Energy Regulatory Commission (FERC) license requirements.

The project is controlled through a microwave / fiber link from Taum Sauk to St. Louis to Osage. The Osage operators operate the Taum Sauk units under the direction of the load dispatcher in St. Louis. As originally designed and constructed, the useable volume in the Lower Reservoir was greater than the volume of the Upper Reservoir. The design volume of the Lower Reservoir was reduced by the need to raise the minimum operating water level from El. 734 feet to El. 736 feet due to the debris being pulled up the tailrace channel. Although trash racks prevent the debris from being pulled into the pumps, the debris still interferes with pumping operations.

Upper Reservoir Dike Construction versus Design

As part of the comprehensive site reconnaissance effort with this forensic investigation, investigators had compared the intent of the original designer as reflected in the construction specifications with post-breach observations. Two major observations evolved from this effort, specifically the fines content (particles of soil size as opposed to sand or gravel) of the rockfill and the preparation of the foundation.

The fines content issue stands out on the slopes of the Breach Area as illustrated below on Figure B.2-16, which is a photo of the north side of the Breach Area. This was supposed to be a rockfill, but it was apparent that fines were left in place and comprise a significant percentage of the overall embankment. It was noted that this was not the original designer's intent and that the shear strength and drainage properties of this finer material acts negatively on the performance of this type of dam and dike construction.

The original construction specifications for the Upper Reservoir were reviewed along with the notes on the Design Drawings and the language of these specifications and notes were compared with observations made during this forensic investigation. A comparison is summarized as follows.

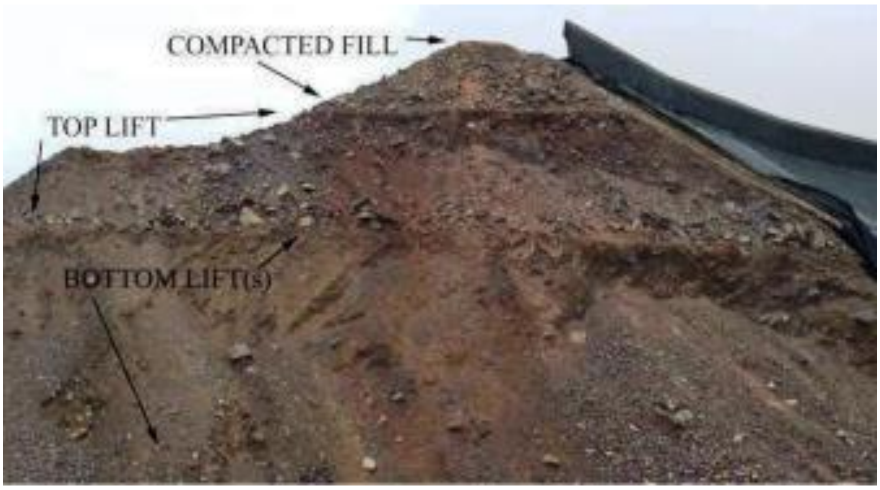


Figure B.2- 16 View of north side of breach area (FERC, 2006)

Specification or Language (1)

"All rock within this area shall be thoroughly sluiced/washed to prevent the accumulation of fines in the upper portions of the face fills."

Post Event Observation (1) *Specification not satisfied*

Specification or Language (2)
"The criterion for determination of adequate sluicing by visual inspection shall be that the entire height of the progressing face and side slopes shall have the appearance of clean rock, and that no areas of concentrated fines, dirt or mud are in evidence."

Post Event Observation (2) *Specification not satisfied. See Figure B.2- 16.*

Specification or Language (3)
"The contractor shall, by control of drilling operations and strength of blasting charge, make every effort to have the rock break into as large pieces as practical for the best construction of the Rockfill Dike."

Post Event Observation (3) *Specification not satisfied. See Figure B.2- 16.*

Specification or Language (4)
"The jetting shall accomplish, principally, the following: a) Wash surface zones of fines away. b) Excavate pockets of spalls and distribute the spalls into voids between larger rocks in contact."

Post Event Observation (4) *Specification not satisfied. See Figure B.2- 16.*

Specification or Language (5)
"Remove topsoil and loose, unstable, altered material as far as possible with bulldozer."

Post Event Observation (5) *Specification not satisfied in all locations. RIZZO forensic investigation borings encountered 18 inches of soil in some locations.*

It was concluded from this comparison that the actual construction deviated significantly and in a <u>negative manner</u> from the intent of the original design. It is contented that this deviation contributed to the instability of the Rockfill Dike in the Breach Area and contributed to the root cause of the Event of December 14, 2005.

Upper Reservoir Dike Design – Construction Practices (1963 versus 2006)
Considering the development of concrete-faced rockfill dams over the last century plus, it is appropriate to understand the differences between modern day (2006) design and construction practices and those followed in the late 1950s and early 1960s when the Taum Sauk Plant was designed and constructed. This comparison bears, to some degree, on the practices to be followed, in the event that the Upper Reservoir is rebuilt. The differences as they apply to the Upper Reservoir Dike design and construction are summarized as follows.

Taum Sauk Design (1)
Rock was dumped and re-positioned by sluicing with water (jets) from monitors.

2006 Design and Construction Practices (1)
Rock is dumped and compacted with heavy compactors and/or heavy tracked dozers.

Taum Sauk Design (2)
Fines were removed by sluicing after rock was dumped into position.

2006 Design and Construction Practices (2)
Fines are removed by screening at the borrow area.

Taum Sauk Design (3)
Foundation was prepared by removal of most deleterious material by dozers. A note on the Drawings that applies to the 70 feet nearest the upstream toe reads is as follows:

> "Strip to sound rock with not more than 2 inches (average) of dirt. This dirt to be thoroughly saturated before placing rockfill."

Another similar note that applies to the center portion of the Dike reads as follows:

> "Remove topsoil and loose, unstable, altered material as far as possible with bulldozer."

2006 Design and Construction Practices (3)
Foundation is prepared by hand labor, water jets, air jets and small excavators. A great deal of detail work, including dental concrete, is often required for concrete or RCC sections.
Bulldozers do not allow for enough detail work and leave too much deleterious material behind.

Taum Sauk Design (4)
Parapet walls were used to retain water on an "everyday" basis.

2006 Design and Construction Practices (4)
Parapet walls are used only if necessary, to act as a short-term barrier against flood levels or wave action. They are not used on an "everyday" basis.

Taum Sauk Design (5)
Grout curtain was installed to a depth of about 20 feet. There is no evidence of the design basis.

2006 Design and Construction Practices (5)
Grout curtains are designed to a depth where rock is essentially impermeable and generally not less than about 30 percent to 40 percent of the sustained head.

In comparing Taum Sauk Design with current (2006) design and construction practices, it was noted that the several deficiencies with respect to the rockfill dam that would be very costly, and perhaps impossible, to repair through an upgrade or remediation. These include excessive fines within the rockfill, inadequate foundation preparation, lack of compaction effort during rockfill placement, and the use of the parapet wall to store water on an everyday basis.

The rebuilt dike of the Upper Reservoir is the largest roller-compacted concrete (RCC) dam in North America (by volume).

Emergency Action Plan

The Emergency Action Plan for the Taum Sauk Project was last reprinted in January 2003 and an annual update was submitted by letter dated August 24, 2005.

Notification Flow Charts: The EAP contains two notification flow charts. Figure B.2-17 is the flow chart for an incident at the Upper Reservoir. There is also another flow chart for an incident at the Lower Reservoir. The Upper Reservoir version calls for immediate notifications of the Johnson's Shut–Ins Park Superintendent and sends plant personnel to warn boaters and campers on the Lower Reservoir. Following these actions, calls are made to the Lesterville Fire Department, Reynolds County Sheriff, AmerenUE employees, FERC staff, and the National Weather Service.

The flow chart for the Lower Reservoir is similar except the Johnson's Shut-Ins Superintendent is not notified since they are upstream of the Lower Reservoir and would not be impacted. The EAP contains the names, addresses, and phone numbers of all residents downstream of the Lower Reservoir that would need to be evacuated from a dam breach. The Lesterville Fire Department would be divided into three teams to notify these residents.

Detection of Emergencies: The EAP explains how an emergency at the project would be detected and evaluated. It was expected an emergency would be detected by (1) first hand observation by plant and security personnel; (2) monitoring of the local and remote instrumentation at the Taum Sauk plant, Osage plant, or load dispatch office; and (3) current weather and new and forecasts obtained from several media sources. The plant and security personnel are on duty from 7:00 am to 6:30 pm and the plant superintendent lives at the project site. It was also noted that abnormal leakage or signs of failure could be observed at Johnson's Shut-Ins State Park by the presence of high or muddy flows.

Upper Reservoir – Inundation Maps: The inundation zone assumed a breach of the west slope of the Upper Reservoir. The failure scenario was initiated by a parapet wall failure leading to breach of the dam. The assumed failure would be preceded by an increase of leakage which would trigger the EAP. About 0.5 hour after the leakage started, the parapet wall fails, releasing 4,500 cfs. It was expected to take about 15 minutes for these flows to reach the flood plain between Highway N and the Johnson's Shut-Ins. In another 0.5 hour, the first slab would breach releasing 14,000 cfs. In the next two hours additional slabs would fail and the reservoir would empty, peak flows would be 30,000 cfs. The final breach would have a bottom width of 60 feet, 1:1 side slopes, and a top with of 240 feet. The peak flow estimate of 30,000 cfs is comparable to the flood of record for the East Fork of the Black River which occurred in 1986. It was expected that the Lower Reservoir would be able to hold the majority of the breach flows.

The path of the flows from the breach is divided into four sections: (1) from the dam, flows would travel 9,000 feet down the wooded/forested slope to the East Fork of the Brown River; (2) flows would travel through the level 5,000 foot flood plain along the East Fork between Highway N and the Johnson's Shut-Ins; (3) flows would travel through a campground and the narrow rock canyon of the Johnson Shut – Ins; (4) flows would pass through 700-foot-long stretch of the East Fork and then enter the Lower Reservoir. The inundation map for the Upper Reservoir does not include arrival times or times to peak flows. There are no developments in the projected floodway from a breach of the north or east sides of the Upper Reservoir. Therefore, the inundation maps do not show a breach in these areas. The flood wave from a breach of the north or east sides would eventually flow into the lower reservoir via the Black River and place the recreational users of the lower reservoir at risk.

Training and Exercises: The project operators received annual training on the EAP. The plant superintendent also performed an annual drill based on a made-up failure scenario and included both licensee personnel and emergency response personnel on the notification flow chart.

Reynolds County Sheriff, Lesterville Fire Department, National Weather Service: The participants were warned of the drill prior to implementing the scenario. The drill was meant to ensure Osage and Taum Sauk Operators acknowledge the alarm and followed their internal procedures and the Taum Sauk superintendent or designee performs notifications according to the postulated emergency. After the drill, the superintendent made follow-up calls to all participants to evaluate the procedures. The last functional exercise for the Taum Sauk Project was per-formed in May 1998. The licensee alternated functional exercises between its Osage and Taum Sauk Projects. A functional exercise was performed at Osage in 2003 and the next functional exercise at Taum Sauk was scheduled for 2008.

Licensee's Account of the December 14, 2005 EAP Activation and Coordination: By letter dated December 27, 2005, AmerenUE provided their detailed account of the EAP detection, activation, and coordination. The following is a paraphrased version. (Ref. FERC, 6)

5:40 a.m.: Plant Superintendent Richard Cooper receives call from Osage operator that they lost indication of the upper reservoir, tailrace, and penstock level transmitter (i.e., the Osage Operator received alarms that reservoir was too low).

6:00 a.m.: Mr. Cooper arrives at project and notices tailrace is muddy. As Mr. Cooper enters powerhouse, he receives call from Lesterville Fire Department reporting flooding at Johnson Shut-Ins. Mr. Cooper informs Fire Department that there are signs the Upper Reservoir has breached. The Fire Department states it will contact the Reynolds County Sheriff, who was currently on another line. Mr. Cooper notified the parties on the notification list, with the exception of the Lesterville Fire Department and Reynolds County Sheriff who were already warned. The Johnson's Shut-Ins park superintendent, Mr. Jerry Toops, is on the notification list, but Mr. Cooper received no answer.

6:30 a.m.: Mr. Cooper completed the EAP contact list. In addition to the contact list, Mr. Cooper also has telephone contact with additional FERC staff, AmerenUE personnel, the U.S. Coast Guard, and Missouri Highway Patrol.

According to a December 27, 2005 letter, AmerenUE states there were no significant problems with implementing the EAP.

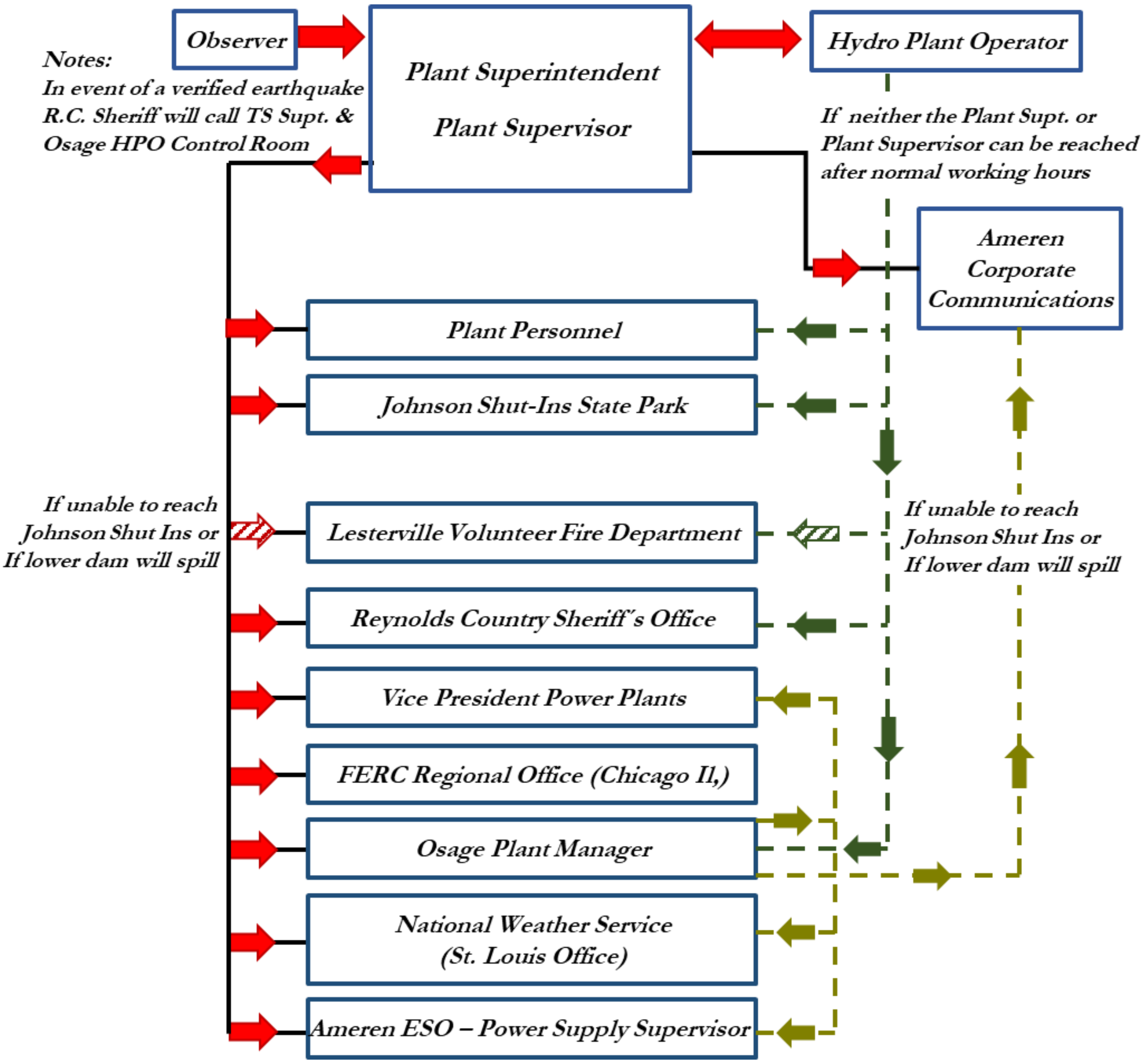


Figure B.2- 17 EAP Notification Flowchart. (Adapted Emeren UE)

B3-EL Guapo Dam

Abstract

El Guapo dam provides water for drinking to an estimated population of 400,000 people and for irrigation purposes. It also provides for flood control. It is located about 4.50 km upstream (south) of the population of El Guapo in the central and northern area of Venezuela. On December 16th, 1999, El Guapo dam failed as a result of the occurrence of an extraordinary hydrological event that exceeded the ability of the spillway and the energy dissipator to handle/pass it resulting in a backward erosion effect that generated a breach in the dam which caused the eviction of more than 120 Mm^3 in less than 30 minutes, causing significant flooding, loss of certain homes, bridges and other minor infrastructure works and, unfortunately, an undetermined number of human lives believed to exceed 400 people. The dam's failure is attributed mainly to the response to the unexpected flood and to the malfunction of the conveyance structure and energy dissipation components that were subjected to a flow much above the design flood that caused the general failure of the dam.

El Guapo Dam Failure (Hidroven 2000)

The Fernando Trías or El Guapo reservoir is located in Miranda State in the north-central region of Venezuela on Guapo River (Figure B3-1) at a site called La Guamita. It was built in order to supply water for drinking to the region of Barlovento for the population of Caucagua, Higuerote and Río Chico (about 400,000 inhabitants) as well as to supply water for irrigation to an area of 6,500 hectares, controlling the floodwaters of the river and maintaining the ecological conditions of the Laguna de Tacarigua National Park.

The original design of the dam was carried out in 1975 by Eng. Manuel Isava for the **Instituto Nacional de Obras Sanitarias** (INOS) and its construction took place between 1977 and 1980.

The dam of El Guapo is located in Barlovento area of Miranda State, 100 kilometers east of the city of Caracas. Since the 1980s, the reservoir has been the most important source of water in the Barlovento System of the state-owned drinking water service company HIDROCAPITAL.

Figure B.3- 1 El Guapo Dam Location (Google Earth, 2020)

The climatic event that occurred in Venezuela during the days 12th to 16th December 1999 generates a flood of such importance in the Guapo river basin that spillway and outlet works are insufficient to handle the volume of the flood and cause the failure of energy dissipator and lateral erosion in the chute generating a breach that culminates in the rupture of the body of the dam even before the overtopping of the dam has occurred.

The main characteristics of the dam and its reservoir are:

Dam		**Reservoir**	
Crest Elevation (masl)	107.00	Total Volume (Mm^3)	141.00
River Bottom Elevation (masl)	47.00	Conservation volume (Mm^3)	129.00
Dam Height (m)	60.00	Dead or Inactive volume(Mm^3)	12.00
Crest Length (m)	524.00	Normal water level (masl)	101.00
Crest Width (m)	8.00	Maximum water level (masl)	103.62
Freeboard (m)	3.38	Minimum water level (masl)	65.00
Total dam volume (m^3)	2,570,000.00	Reservoir area at NWL (ha)	600.00

Principal Spillway		**Secondary Spillway**	
Type: Ogee free	Uncontrolled	Type Tunnel	Controlled
Crest elevation (masl)	101.00	Section	Horseshoe
Width (m)	12.00	Dimensions (m)	3.00 x 3.00
Chute Length (m)	282.00	Length (m)	400.00
Chute Width (m)	12.00	Gate dimensions (m)	H=6.00 W=3.00
Slope (m/m)	49.68%	Crest elevation (masl)	95.00
Capacity (m^3/s)	101.80	Capacity estimated (m^3/s)	115.00
Freeboard (m)	3.38		

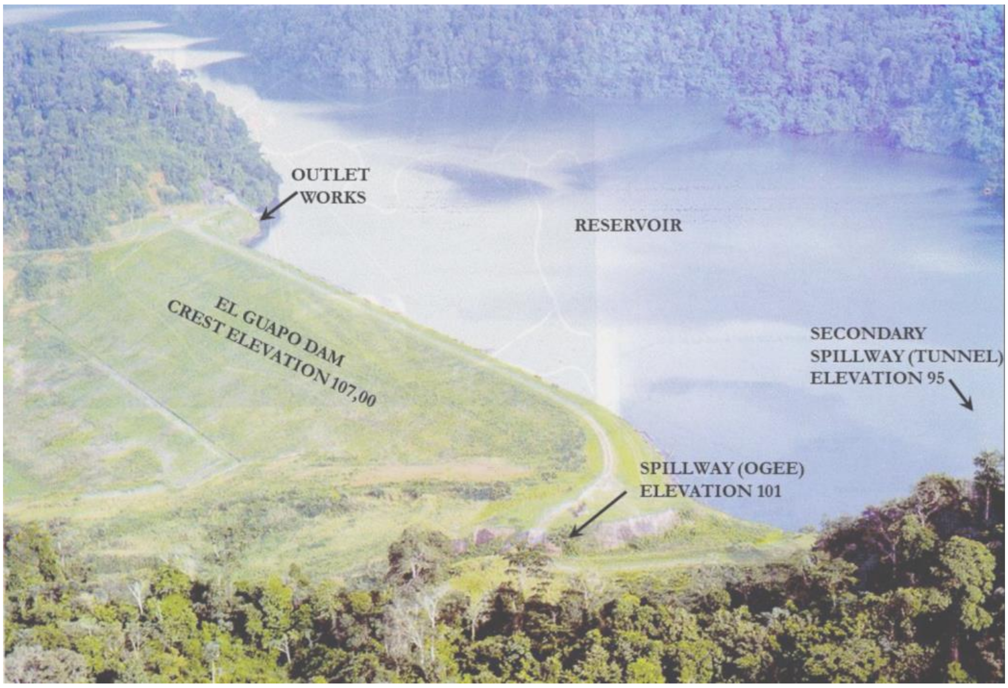


Figure B.3- 2 El Guapo Dam (Hidroven)

Background

The incidents related to the El Guapo dam and the floodwaters of the river have been very diverse throughout its short operational life and, as a result, on several occasions they had to do expansion work on the spillways and outlet works. During the design phase, the hydrological estimates related to floods were made based on measurements in the neighboring basins due to the absence of rainfall and fluviometric data typical of the basin and the river under study. Additionally, all the studies carried out were based on the definition of maximum discharge flood without taking into account specific flood events that maximize the volume traveled by the reservoir.

The design was carried out in 1975 and its construction between 1977 and 1980. During the construction of the dam many problems were encountered; some of them being, the displacements in the left abutment due to excavation and the occurrence of a flood during construction that affected the diversion tunnel due to the large amount of vegetable debris waste transported by the water as well as the change in the design of the energy dissipator that had to be rebuilt.

The magnitude of the flood that occurred during construction necessitated review of the original hydrological study and, as a consequence of this analysis, the need to design an auxiliary spillway, called Tunnel Spillway or Secondary Spillway, was felt. The secondary spillway began operations in 1985 and remained under regular operations until December 16, 1999.

The design establishes the convenience of operating spillways structures under the following operating scheme:

- Whenever the reservoir exceeds the elevation of 95.00 meters above sea level, the secondary spillway must be opened and the bottom discharge located in the intake tower.
- These structures will remain open even when the water reaches level 101 and the discharge of water through the main spillway begins.

The maximum discharge capacity of the elements available for the design condition and that occurred on December 16, 2019 are shown in Table B.3-1.

Reservoir (Elevation)	Spillway (Ogee) m^3/s	Secondary Spillway (Tunnel) m^3/s	Outlet work Bottom discharge m^3/s	Total Discharge m^3/s
Design (103.62 m)	**101.80**	**87.75**	**23.00**	**212.53**
December, 16th (106.80 m)	**335.20**	**99.80**	**23.59**	**458.64**

Table B.3- 1 Estimated discharge capacity in spillway and outlet works

December 1999

As already mentioned, the month of December 1999 was extremely humid and constitutes one of the most unique hydrological events in the hydrological history of the country; however, there are few reliable data regarding the amount of water that precipitated during those days, a circumstance that prevents a thorough analysis of what happened.

During the first 11 days of December, the El Guapo reservoir was relieved by the secondary spillway maintaining the reservoir levels between 95 and 101 meters over sea level; However, on December 12th there is a first flood that contributes about 32 million cubic meters and raises the reservoir level 3.35 meters in one day. Subsequently, on December 13th for 12 consecutive hours (6:00 am to 6:00 pm), the reservoir remains practically stable (elevation 102.09), and the outlet works discharge an average of 104.50 m^3/s, so that, during on the night of the 13th and practically all the 14th of December, there is a reduction in the water level and an increase in the freeboard of the reservoir. During these first 24 hour (12/13 6:00 p.m. to 12/14 6:00 p.m.), the level of the reservoir decreases until reaching the 101.82 level. (See figure B.3-3).

A gap of important information occurs during the period between 6:00 pm on the 14th and 2:50 pm on the afternoon of the 15th at which time the reservoir level is 1.58 meters above the Maximum Water Level expected (elevation 105.80) with a total discharge exceeding 320 m^3/s higher than the design flows for the Design Flood. The maximum level recorded occurs on December 16 at 1:00 am in the morning when the reservoir is located at a height of 106.70 just 30 cm below the crest level.

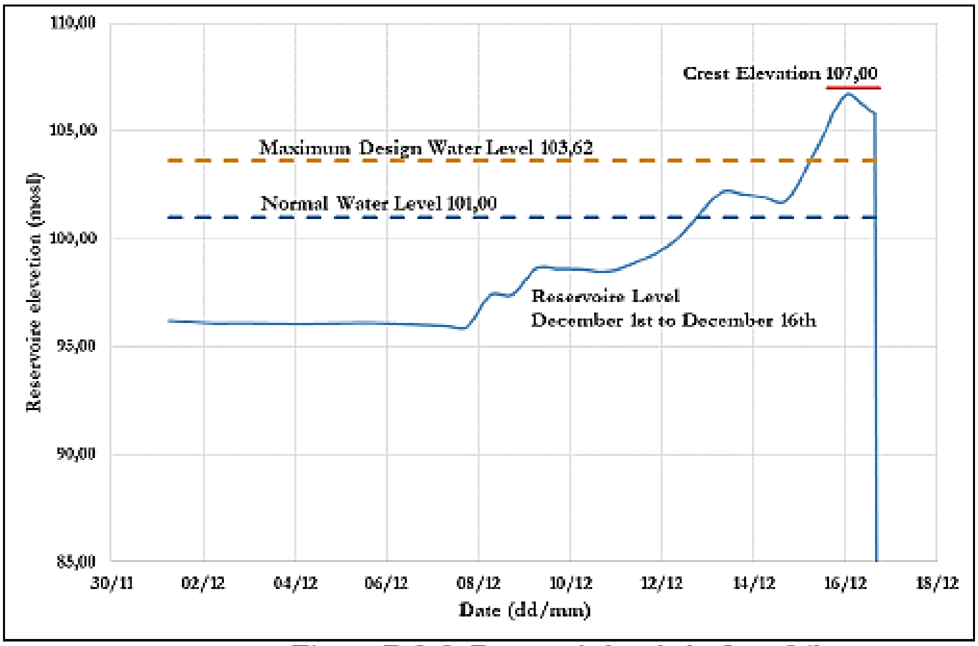


Figure B.3- 3 Reservoir levels before failure (Adapted L. Rotundo, 2000)

On Thursday, December 16th, spillways were subjected to flows that tripled their design condition so that, finally, between 4 and 6 pm a opens a breach of such magnitude that allowed the discharge of about 140 million of cubic meters of water in a period that some set between 30 and 90 minutes.

The consequences of the failure of the El Guapo dam can be summarized as (see also Figure B.3-4):

- More than 7,375 people affected and unquantified human losses that according to the source of information are between 50 and 400.
- Seven (7) missing towns and 13 more affected.
- 790 homes destroyed 1,500 homes affected and unquantified material losses.
- Losses of 60% of agricultural land cultivated at the time of the collapse of the dam.
- Destruction of 2.5 kilometers of National Roads and collapse of three (3) bridges.
- Loss of the source of water supply of about 400,000 inhabitants of the Barlovento area for a period of six (6) years.
- Major effects on electricity distribution networks, water supply networks and sewerage.

Figure B.3- 4 Effects of the failure of El Guapo Dam. (L.Díaz, 2015)

El Guapo Dam Failure

The failure of El Guapo is a very interesting lesson about the effects that the underestimation of the hydrological response of the basin can cause during extreme events.

The failure occurs due to an inability of the dam and its spillway to handle the large volumes of water that entered the reservoir and totaling 145 Mm^3 in a period of 16 days but concentrating on 117.81 Mm^3 (equivalent to 90% of the active volume of the reservoir) in a span of 129 hours that can be seen as a constant contribution of the river of 253.70 m^3/s.

According to the analysis of extreme floods in Venezuela (Córdova; 2006) it can be concluded that:

> *"The main problems associated with the estimation of the hydrograph of the millenary flood selected for the design of the spillway, in the original project, is the following:*
>
> - *Assume that the extreme flows follow a distribution of type I or Gumbel extreme events.*
> - *Overestimation of losses or abstractions, with the consequent underestimation of the effective histograms associated with different return periods.*
> - *Lack of calibration of the flood model.*
> - *Do not consider the occurrence of complex floods, where the volume of the same becomes as important as the maximum flow of the associated hydrograph "*

It should be noted that the almost total absence of hydrological data in the basin analyzed contributed to the impossibility of making precise adjustments to the results obtained and the correct calibration of the models used. In this same study Córdova shows Figure B.3-5 where it is possible to appreciate the great difference that exists between the results obtained in the project phase and in this review.

The Failure Process

The photographic sequence taken during the day of the fault shows three (3) relevant aspects when analyzing the fault itself: The Energy Dissipator; the Chute of the Spillway and the formation of the Breach that makes the dam fail without overtopping the dam.

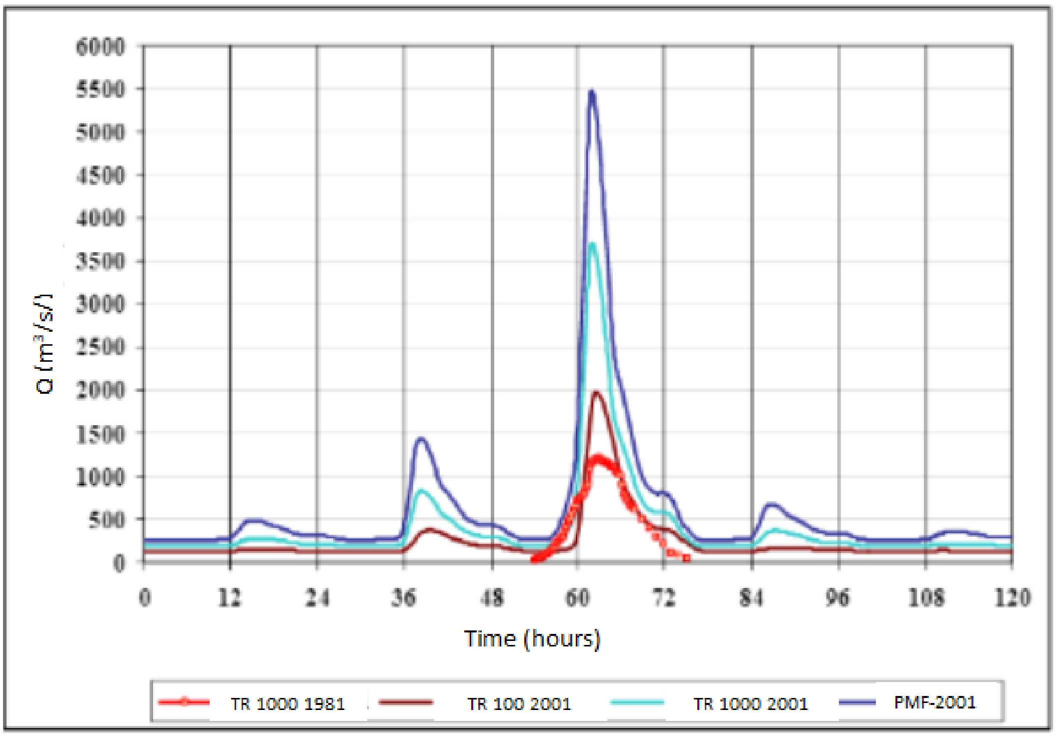


Figure B.3- 5 Río Guapo Hydrograph comparison (Córdova, 2007)

The Energy Dissipator

According to the description of the original design (Isava, 1975), is a Type I stilling basin of the USBR of 26 meters in length and a height of walls of 7.50 meters in height that confines the maximum conjugate depth expected of 6.50 meters. This situation contrasts with the hydraulic jump that is generated during the occurrence of the 1999 flood that can be seen in Figure B.3-6 where the stilling basin acts as a flip bucket. The erosion process is also observed behind the walls of the dissipator.

Figure B.3- 6 Operation of the Energy Dissipator (Hidroven, 2000)

The Chute of the spillway was overtopped in almost its entire length and the flood that exceeded its capacity generated lateral erosive effects that initiated a process of dam failure by developing a backward erosion process in the contact of the dam and the left abutment. The estimates made taking into account the reported levels of the reservoir during its last days of operation, establish that the spillway flow exceeded its maximum design capacity for a continuous period of not less than 25 hours reaching a flow of more than 300 m^3/s; that is, 210% higher than design discharge.

Figure B.3-7 to B.3-9 shows various views of the spillway operating about 2 hours before the collapse of the dam.

Dam failure occurs with the formation of a breach that joins the backward erosion process generated around the dissipator with undermining erosion that occurs below the chute due to the overflow of its lateral walls.

In an attempt to support spillway release and the volume of water that arrived at the dam, the bottom outlet of the project was opened on December 15 at 6:00PM. Although this situation constitutes an operation not in accordance with the original operation manual, since it had to be opened from the moment the reservoir reached level 95, the low contribution of the bottom discharge does not constitute a fundamental factor in the overtop of the chute and, therefore, in the failure of the dam (L. Rotundo, 2000).

Figure B.3- 7 El Guapo Dam Chute – December 16th, 1999. (Hidroven, 2000)

Figure B.3- 8 More Views of El Guapo Dam Chute – December 16th, 1999. (Hidroven, 2000)

Figure B.3- 9 Secondary spillway discharge bottom outlet El Guapo Dam – December 16th, 1999. (Hidroven, 2000)

Figure B.3- 10 El Guapo failure. (Hidroven, 1999)

Figure B.3- 11 El Guapo Dam hydraulic model. (EDELCA, 2002)

El Guapo Dam – Rehabilitation

The rehabilitation of the El Guapo Dam begins with the review of hydrological study of the basin and the definition of the new design floods both from the point of view of maximum discharges and that of complex floods of maximum volumes such as occurred. The maximum expected flow for the maximum probable flood (CMP) is 5,487 m^3/s which contrasts with the 672 m^3/s that corresponded to the 1,000-year return period of the original design flood.

A hydraulic model investigation was carried out by EDELCA in the Hydraulics Laboratory of this institution in Puerto Ordaz, Venezuela in 2002 and the execution works are carried out on an emergency basis.

The construction process begins with the river diversion work between 2002 and 2003. The diversion was carried out with a tunnel 300 meters long and 4.40 meters in diameter, a 16 meter high coffer dam and a berm that prevented the flow of water in the work area. Figure B.3-12 shows the entrance structure of the diversion work and the internal section of the tunnel. Upon completion of its function as a diverting element, this work was modified to be operated as a bottom outlet by installing regulating valves at its lower end.

Figure B.3- 12 Bypass Tunnel El Guapo Dam (Dell'Aqua C.A., 2003).

The replacement of the dam in the space eroded by the river between the left abutment and the old dam is carried out with a roller compacted concrete structure (RCC) being this the first experience of this type in Venezuela.

The RCC dam is 138 meters and 50 meters high using a total of 330,000 cubic meters of compacted concrete, 36,500 cubic meters of conventional concrete and 810,000 cubic meters of earthwork for its execution.

The work is developed by the company Camargo Correa between August 2005 until February 12th, 2009, date on which it was formally commissioning.

The RCC dam has an upstream slope of 0.60H: 1V while the downstream slope is 0.85H: 1V. The new spillway has the capacity to evacuate up to 2,700 m^3/s, a width of 43 meters and the relief level had to be lowered to level 97 in order to preserve the rest of the hydraulic components of the original dam.

Figures B.3-13 courtesy of the construction company shows various phases of the construction process of the RCC dam and in Figures B.3-10 and B.3-11 shows the fully finished dam, during the filling process of the reservoir and, later, during normal operation.

Figure B.3- 13 El Guapo Dam – Reconstruction Process. (Camargo Correa, 2005-2008).

Figure B.3- 14 El Guapo Dam – after Rehabilitation. (C. Hernández, 2008)

Lessons learned

- The safety of the dams is directly associated with the correct estimation of the extraordinary floods that may occur in the intervened basin. The hydrological analysis should consider maximum discharges as complex floods with maximum volumes.
- Spillways should be designed for the flood established according to the importance of the Dam and the specific conditions of each project but its operation should be reviewed for the maximum conditions that the reservoir may require in case hydrological estimates are exceeded.
- The conditions for the foundation of spillways including chute and stilling basin must be competent and able to withstand temporary erosive processes. In case of not having

these conditions, improvement and protection works must be carried out in the adjacent soils.

- It is essential to have supervision and monitoring systems for outlet works and spillways in dams and to have contingency plans and general knowledge that allow an early response to extraordinary and catastrophic events.
- The dam's operating manuals must be strictly compliant and must be easily accessible to all those involved in the ordinary operations.
- Constant hydrological measurements, both pluviometric and fluviometric, in the intervened basin as well as in the rivers that feed the reservoir is a task that will improve the knowledge of the rivers and adjust any distortion that could have the design simulations as well as analyze possible modifications in rainfall patterns and river hydrological responses as a result of climate change or anthropic processes of river basin alteration.
- It is very important to have access to the dam area comfortable and safe at all times that facilitates the operation, inspection and monitoring of the main structure and all its appurtenance works

Figure B.3- 15 El Guapo Dam – operation after rehabilitation. (Camargo Correa, 2009)

B4-Spencer Dam failure – March 13, 2019

Abstract

The Spencer Dam collapse has been considered as the first in the U.S. caused by ice-jam, and the failure of the dam was age related. Heavy winter precipitation during the March 2019 North American blizzard (snowstorm) led to the devastating and historic floods and caused major damage across the Plains, Midwest, and Mississippi River corridor. This can all be attributed to the wetter than average end to the year 2018 along with a very wet start of 2019. Not to mention, snow pack that was just beginning to melt in these locations. Nebraska's Niobrara, Platte, Missouri and Elkhorn Rivers broke their banks in the week of March 13, 2019, and the Niobrara River broke the Spencer Dam submerging the nearby town of Lincoln. Operators attempted to open the manually-operated gates of the dam to release floodwater, but some were frozen shut by the cold weather and might have contributed to water overtopping the earthen embankment. According to the authorities, the Spencer dam was compromised sometime after 5 PM on Wednesday, March 13, 2019. The collapse of the 26-foot-high dam unleashed a wall of water 11 to 15 feet in height carrying large ice-chunks the size of cars, washing away a home and its owner, several trailers and a unique straw-bale saloon/bait shop. Emergency evacuation and post-failure investigation were carried out. The figures given below show before and after failure of the Spencer dam.

Spencer Dam after failure (Hydroreview.com, 2020)

The Spencer Dam failure incident

Figure B4- 1 The pre-failure Spencer dam.

Figure B4- 2 The post-failure Spencer dam.

Figure B4- 3 Overwhelming ice and water breached the Spencer Dam at 5:30 am, 2019.

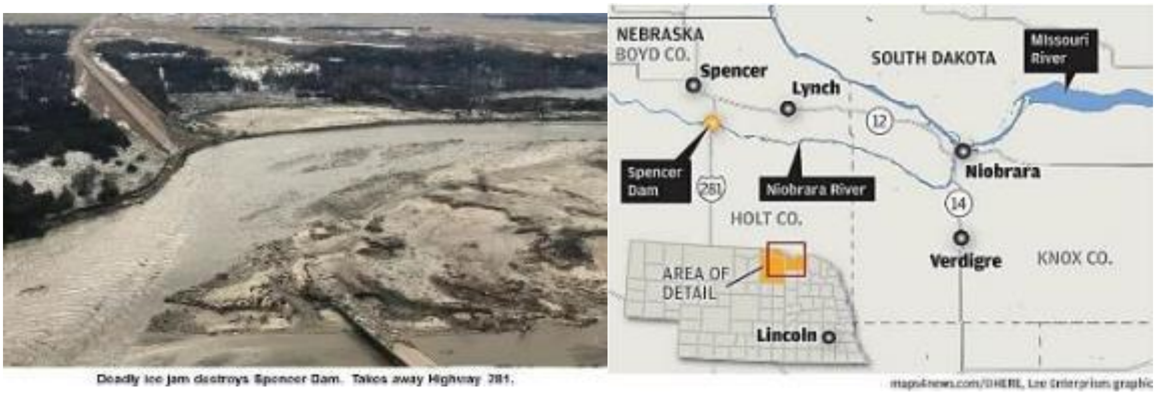


Figure B4- 4 The ice jam took away Highway-281 bridge. Location map of the Spencer dam (maps4news.com).

In the winter months of late 2018 and early 2019, devastating historic floods caused major damage across the Plains, Midwest, and Mississippi River corridor. Nebraska‟s Niobrara, Platte, Missouri and Elkhorn Rivers broke their banks in the week of March 13, 2019, and the Niobrara River broke the Spencer Dam (see Figure B4-5 below) submerging the nearby town of Lincoln. According to the National Weather Service and the Nebraska State Patrol, the Spencer dam was compromised sometime 5 PM on Wednesday, March 13, 2019. The collapse of the 26-foot-high dam unleashed a wall of water 11 to 15 feet in height carrying large ice-chunks the size of cars, washing away a home and its owner, several trailers and a unique straw-bale saloon/bait shop.

On the cold morning of the following day, Kenny Angel, who resided in a house close to the saloon, got a frantic knock on his door. Two workers from a utility company in northern Nebraska had come with a stark warning: Get out of your house. Water was coming over the top of a nearby hydroelectric dam, they told him before fleeing in their truck. Minutes later, the swollen Niobrara River crashed through Spencer Dam, unleashing a wave of water with ice chunks. Angel's home was wiped away; his body was never found. “He had about a 5-minute notice, with no prior warning the day before,” Scott Angel, one of Kenny's brothers, said. One other male victim was rescued from his house by the O’Neill Fire Department using a boat and rope rescue equipment after he was found trapped by flood waters.

The Spencer Dam collapse may be the first in the U.S. caused by ice-jam.

Spencer Dam project

Figure B4- 5 The Spencer dam and the powerhouse before failure (looking upstream).

The Spencer Dam is located on the Niobrara River, a tributary of the Missouri River that runs 568 miles from Wyoming, through northern Nebraska. The river drains snow melt from the eastern Rocky Mountains as well as one of the most arid stretches of the Great Plains. The dam was 91 years old in 2018 and was a run of the river hydroelectric project on the Niobrara River in Boyd County and Holt County, Nebraska, about 5 miles (8.0 km) southeast of Spencer. The dam was operated by the Nebraska Public Power District (NPPD).

Initial construction was completed in 1927 (see Figure B.4-6), at a site located 39 miles (63 km) upstream of the Niobrara's confluence with the Missouri River. The dam was originally 3,698 feet (1,127 m) long and 26 feet (7.9 m) high, constructed in several sections. The powerhouse was located on the north bank of the river, consisting of two Westinghouse generators with capacities of 2,000 and 1,300 kilowatts. The spillway consisted of four tainted gates and five stop-log gate elements to the south of the powerhouse. The combined length of the powerhouse and spillway was 404 feet (123 m); the rest of the dam was an earth embankment extending to the south side of the Niobrara valley. The dam originally formed a reservoir with a gross capacity of 16,847 acre feet (20,780,000 m^3), a live storage of 5,306 acre feet (6,545,000 m^3), and a surface area of 1,200 acres (490 ha). In 1935, the dam was partially breached after an ice jam broke upstream. It was reconstructed in 1940 (see Figure B.4-7).

Both after its initial construction and after rebuilding in 1940, the dam/reservoir was filled with sediment within a few years. Therefore, Spencer Dam functions as a sediment trap and the 486 ha reservoir requires sediment flushing each spring and fall. Although this did not affect hydropower generation since this hydroelectric project was operated to match the river flow, twice-annual sluicing had been conducted since 1948 to clear sediment away from the power intakes. The dam gates were opened for about two weeks each year, allowing the river to flow through and wash accumulated sediment downstream. This stretch of river is used by several endangered species, including pallid sturgeon, interior least tern, piping plover and whooping crane. Between 1975 and 1989, fish kills occurred in the Niobrara River that was attributed to sluicing activities. In response, engineers began draining the reservoir more slowly, which has worked in preventing further fish kills.

By 1985, Spencer Dam was the only active hydroelectric plant on the Niobrara River, after the upstream Cornell Dam hydroelectric plant ceased operation. The NPPD had rights to 1,400,000 acre feet (1.7×10^9 m^3) of water per year for power generation. As of 2007, the average annual flow through the power plant was 874,000 acre feet (1.078×10^9 m^3). In 2007, due to severe declines in the flow of the Niobrara River as a result of upstream irrigation, NPPD requested that the state of Nebraska make farmers pay as compensation for lost power generation. Several lawsuits were filed as a result, but as of May 2010 all had been decided in favor of NPPD, whose water rights are older than the irrigators' rights.

In September 2015, NPPD announced that it would be decommissioning the Spencer hydropower plant in two years due to increasingly uneconomical cost of power generation at this site. The water rights would be sold for $9 million to the Nebraska Game and Parks Commission and five local natural resources districts to manage the river for agriculture, recreation and wildlife conservation.

Nebraska Public Power District shut down its hydropower plant at Spencer Dam on the Niobrara River in 2017 and gave up its water rights on the popular canoeing stream. It cleared the way for the Nebraska Game and Parks Commission and a coalition of five local natural resources districts across North Central Nebraska to get the right to the water for recreation, conservation and continued irrigation.

Emergency Actions

State inspectors had given the dam a "fair" rating less than a year earlier than the failure time. Until it failed, it looked little different from thousands of others dams across the U.S. – and that could portend a problem.

Before the dam was overtopped, NPPD workers had attempted to open the manually operated gates of the dam to release floodwater, but some were frozen shut by the cold weather and may have contributed to water overtopping the earthen embankment a short while later

Shortly after the dam burst, Knox County Sheriff announced: "The Knox County Sheriff's Office has been advised that the Spencer Dam has been compromised. We are trying to contact everyone along the Niobrara River to evacuate them. Please pass the word. Niobrara and Verdigris (Nebraska) Fire and Rescue are on stand-by."

All emergency personnel in Boyd and Holt Counties were activated, and the Nebraska Governor Pete Ricketts declared a state of emergency in 53 counties, saying that one-third of the state has been affected in the devastating floods.

The dam failure forced the evacuation of residents in low lying areas along the river. On a cold morning in March, 2019, Kenny Angel got a frantic knock on his door. Two workers from a utility company in northern Nebraska had come with a stark warning: Get out of your house. Water was coming over the top of a nearby hydroelectric dam, they told him before fleeing in their truck. Minutes later, the swollen Niobrara River crashed through Spencer Dam, unleashing a wave of water carrying ice chunks the size of cars. Angel's home was wiped away; his body was never found. "He had about a 5-minute notice, with no prior warning the day before," Scott Angel, one of Kenny's brothers, said.

One male victim was rescued from his house by the O'Neill Fire Department using a boat and rope rescue equipment after he was found trapped by flood waters.

Another male victim, Kenneth Angel who resided in a house close to the saloon, is believed to be lost to the flood. Search and rescue efforts were hampered by inclement weather conditions and rapidly moving flood waters created by the breach in the dam. The search effort continued as weather conditions improved and as more resources became available.

Mandatory evacuations were underway in communities downstream of the dam in Boyd and Knox Counties. Reports said the swelling river had taken out the bridge next to the dam on Highway 281, the Highway 11 Bridge south of Butte, and the Redbird Bridge south of the Niobrara in Holt County.

Resulting from record flows on the Niobrara River, the Chief Standing Bear Memorial Bridge across the Missouri River below the Niobrara confluence was temporarily closed as the Nebraska approach flooded, but it reopened a few days later.

Historic Flooding

The river drains snow melt from the eastern Rocky Mountains as well as one of the most arid stretches of the Great Plains. In the winter months of late 2018 and early 2019, devastating and historic floods had already caused major damage across the Plains, Midwest, and Mississippi River corridor this winter. This can all be attributed to the wetter than average end to 2018 along with a very wet start to 2019. Not to mention, snow pack that was just beginning to melt in these locations.

The failure of Spencer Dam can be attributed to the high water and rapid snow melt occurring in the Great Plains, mix with previous week's blizzard, and things would not go according to plan.

There were enough water and wind to wash out the dam causing mass flooding and an 11-foot "wall of water" to flow downriver.

Figure B4- 6 Spencer Dam as it appears in its rebuilt condition in 1945, after a 1936 failure of the dam's overflow weir, 1945.

Figure B4- 7 Spencer dam soon after it was built around 1927.

Post Failure Investigation

The initial break of the dam destroyed a saloon and private residence below the dam and washed-out part of U.S. Highway-281-bridge. The owner of the house was reported missing and presumed dead. In the town of Niobrara, dozens of buildings and the Mormon Canal Bridge carrying Nebraska Highway 12 were washed away. A water pipeline under the Niobrara River bed was destroyed, cutting off the supply for about 2,000 people in Boyd County.

> *NPPD later said that workers had attempted to open the dam's manually operated gates to release floodwater, but some were frozen shut by the cold weather and may have contributed to water overtopping the earthen embankment a short while later.*

In addition, "truck-sized ice chunks" were reported to have hit the dam and caused the destruction of the bridges downstream. During an inspection in April 2018 the Nebraska Department of Natural Resources had warned that "deficiencies in the dam exist which could lead to dam failure during rare, extreme storm events."

In the early morning of March 14, 2019, the dam was breached after a major storm caused heavy rain, snowmelt and ice breakage that in turn caused swelling of the Niobrara River. The earth embankment was washed out in two locations, while the spillway remained partially intact (though heavily damaged). An 11-foot (3.4 m) wall of water was released by the failure, as recorded by a U.S. Geological Survey stream gage moments before it was washed away.

According to an inspector, the late winter ice-jam on the river played a leading role in the collapse of Spencer Dam. Some giant ice chunks weigh as much as a full-sized pickup truck.

State inspectors had given the dam a "fair" rating less than a year earlier. "With age comes problems with any infrastructure – steel corrodes, concrete deteriorates over time," Tim Gokie, chief engineer of the state's dam safety program, said. "But there's no indication that any of that led to the failure of the dam."

After the failure, the powerhouse of the dam remained standing and a small portion of the bridge on the Boyd County side of the river is also visible (see Figure B.4-8 to B.4-11). The approach and most part of the bridge are gone on the Holt County side of the river. Upriver to the west, the bridge north of Stuart crossing the Niobrara River was also destroyed due to the flooding following the Spencer dam breach.

Figure B4- 8 Rain and snow melt take out Spencer Dam, 2019.

Figure B4- 9 Spillway and powerhouse of Spencer Dam, 2019.

Figure B4- 10 Before and after of Spencer Dam´s powerhouse failure, 2019. (ASDSO, 2020)

Figure B4- 11 Looking north back towards the powerhouse of the Spencer Dam, 2019.

Figure B4- 12 Spencer Dam, 2008. (Hydroreview.com)

B5-Maneri Dam - Repair of Spillway and its Energy dissipater severely damaged by rolling boulders during floods

Abstract

Maneri Bhali Stage-I (90 MW) Hydroelectric scheme was planned to harness the energy of waters flowing down river Bhagirathi between Maneri and Uttarkashi. It was commissioned in October 1984. The annual generation of this project is approximately 455 MU.

Features of Maneri Dam:

- A 39 m high and 127 m long Concrete Gravity Dam across river Bhagirathi near Maneri, which houses the spillway. The spillway designed to pass 5,000 cumec. It consists of four numbers radial gates of size 13 m width & 14.55 m height each, separated by 4 m thick piers. A slotted roller bucket provided for energy dissipation. Downstream view of Maneri dam can see in Figure B.5-1.
- An Intake structure, which comprises of three bays 9.00 m. wide each with an all-weather channel on the left flank. A sedimentation tank with 08 hoppers of 15.00m width, 15.70 m length and 5.75 m depth in two rows (i.e. 4 in each row) provided to remove silt particles, which flushed back to the river downstream of the dam through a silt-flushing tunnel.
- An 8,631 km long and 4.75 m diameter circular concrete lined Head Race Tunnel.
- A 69 m high and 11 m diameter underground surge shaft of restricted orifice type along with 316 m long and 6m diameter upper expansion chamber, 89.5 m long and 6m diameter lower expansion chamber.
- About 456 m long steel lined penstock of 3.8 m diameter with three branches of 2.5 m diameter just upstream of the powerhouse.
- A surface powerhouse near Uttarkashi, housing three Francis turbines of 30 MW capacities each, i.e. total installed capacity of 90 MW. The firm power is 38.23 MW. The design discharge of the power station is 71.4 m^3/s. The difference in elevation between the dam and the power station provides for a design head of 147.5 meter and a gross head of 180 meters.
- An open tailrace channel about 120 m long joins river Bhagirathi at Uttarkashi.

Figure B.5- 1 Downstream view of Maneri Dam. (CWC)

Details of damages

Severe floods in the rivers of Uttarakhand in August 2012 and June 2013 badly damaged some hydroelectric power projects mainly in Bhagirathi and Alaknanda river valleys. The spillway of Maneri Bhali Stage –I dam was also severely damaged during these floods with heavy leakages downstream from the spillway gates.

The damages observed in Spillway bays no. 1 & 2 on the right bank (Figure B.5-2) were mainly as under: -

a) The sill beam of stop log gate and radial gate was washed out in a length of approximately 6.00 m and 8.00 m respectively out of a total length of 13 m. The flange of the remaining sill beams was also completely damaged.

b) As both the sill beams were washed out in some length, therefore a cavity of about 6 - 7 m in length along the piers having a depth up to 2.00 m was created in between the radial gate and stop log gate. The flood also damaged the spillway glacis on the right side and in the d/s of radial gate sill beam throughout its length & up to the slotted roller bucket in about 3.00 m width with varying depth with a maximum 13.00 m depth.

Figure B.5- 2 Shows the damages in Spillway bay no-1

The damages observed in Spillway bays no. 3 & 4 on the left bank were mainly as under:

- The concrete in between stop log gate and the radial gate eroded due to high flood. The depth of erosion was in the range of 0.25 m to 0.70 m.
- The spillway glacis also damaged. The depth of erosion ranged from 0.25 m to 0.80 m.

The damages in the energy dissipation arrangement and the downstream areas were mainly as under:

- The slotted roller bucket completely damaged and washed out (Figure B.5-6).
- Deep scour holes/cavities observed in the bucket area (Figure B.5-7).
- The spillway training walls badly damaged – the damages being much more in the right training wall (Figure B.5-6).

Probable causes of damages

A close examination of the flow conditions in Maneri Bhali Stage-I dam reveals that the following were the major reasons of damages

- Maneri dam silted up to the spillway crest. Because of this, big size boulders, which come with the floods in monsoon period, roll down the spillway and cause damages to the spillway glacis, bucket and downstream training walls of Maneri dam.
- In the upstream side, there is a turn in the river and shortest radius of the turn is towards the bay no.1 on the right bank. Because of this, rolling boulders in monsoon season pass more through bay no. 1 in comparison to other spillway bays resulting in more damages in the bay.
- The width of the river downstream of the slotted roller bucket is converging. The training walls obstruct the flows released downstream from the spillway resulting in adverse flow conditions d/s of bucket/return flows with eroded materials drawn into the bucket etc. and results in damages to spillway structure and roller bucket.

Rehabilitation works carried out by UJVNL prior to DRIP in Spillway bays No. 1 & 2

Initially to arrest leakages, temporary arrangements such as caulking of gates were undertaken. After the monsoon of 2014, comprehensive rehabilitation/repair of Maneri Dam was planned by UJVNL. First of all rehabilitation works for the spillway bays no. 1 & 2 on the right bank were undertaken. The repair works of these spillway bays carried out on priority basis in two phases:

- Repair work from the sill beam of stop log gates to 2.00 m downstream of sill beam of radial gates by taking shutdown of Power House for a period of 36 days from 02.12.2014 to 07.01.2015.
- Repair work in entire spillway glacis from 2.00 m downstream of sill beam of radial gate in running powerhouse condition.

Repair works from the sill beam of stop log gates to 2.00 m downstream of sill beam of radial gates in spillway bays 1&2.

Following activities performed in sequential order during the closure period to complete the works:

- Creation of bund in front of spillway bay 1, 2 & 3 to divert water through spillway no. 4 (Figure B.5-3).
- Dismantling damaged mild steel sheets previously placed over spillway concrete profile.
- Damaged & remaining sill beams of stop log gate and radial gate removed.
- Dismantling of concrete to get a minimum thickness of 250 mm of new concrete and a cover of 200 mm over the reinforcement.
- Drilling hole of 600 mm depth and 32 mm diameter in a staggered manner in parent concrete for fixing anchors.
- Fixing 25 mm diameter Fe 500 TMT bars in drill hole with grouting material for anchorage of reinforcement as well as dowel bars for fixing of MS plates.
- The reinforcement laid with 20 mm diameter bars @ 150 c/c keeping the top cover of 200 mm. Where the cavity was more than 80 cm deep, reinforcement provided in intermediate layers also keeping the top cover & bottom cover of 200 mm.
- Applying bonding coat (BASF make) between old and new concrete.

- Laying of concrete (ACC make Dry Crete of M80 grade) in the cavity between sill beam of stop log gate and radial gate (Figure B.5-4).
- Fixing sill beams i.e. ISMB 300 (300 x 140) conforming to IS 2062 (E250) with 20mm SS plate for Stop Log gates and Radial gates.
- Laying concrete (ACC make Dry Crete of M80 grade) up to the final level as per the profile.
- Fixing MS sheet (20 mm thick with a yield strength of 450 MPa) over the prepared concrete surface (Figure B.5-5).
- Grouting between steel plate and concrete through holes cut in the steel plate.
- To complete the repair works of spillway bays no. 01 & 02 within stipulated period, all the pre-requisites such as man power, material etc. arranged before the start of work. The works taken up in three shifts continuously and completed within 36 days and the powerhouse restarted thereafter.

Repair Works in Spillway Glacis of Spillway bays 1&2

Repair works in spillway glacis carried out from 2 m d/s of radial gates during powerhouse running condition. All activities were same as stated above. Only instead of M80 Dry Crete, M60 grade concrete was used.

Based on IS 10262-2009 and IS 456-2000 a mix design for M60 concrete (used for repair of spillway bays 1&2) carried out by UJVNL, which is as under:

a)	Cement	450 kg/m^3
b)	Water	157 kg/m^3
c)	Silica fume	25 kg/m^3
d)	Fine Aggregate	598 kg/m^3
e)	Coarse Aggregate	1,269 kg/m^3
f)	Chemical Admixture	7,125 kg/m^3

The 28-day target strength of trial mix proportions for M60 grade of concrete achieved as 68.9 N/mm^2.

Figure B.5- 3 Temporary Bund for channelization of stream water

Figure B.5- 4 Laying concrete in spillway bay no 2.

Figure B.5- 5 Laying and Grouting of MS Plate in Spillway bay no 2.

Figure B.5- 6 Damaged Left Training wall (CWC, 2017)

Figure B.5- 7 Cavity in Slotted Roller Bucket (CWC, 2017)

Rehabilitation/repair works carried out/proposed under DRIP

1. *Re-alignment and Reconstruction of downstream training walls*

The training walls originally constructed with a converging alignment in plan. In addition to the slotted roller bucket which was totally damaged, the training walls were also badly damaged in the floods of August 2012 and June 2013 as can be seen in Figure B.5-6. To improve the flow conditions and to mitigate the damages due to flowing boulders, these walls were dismantled and re-constructed with a straight alignment. A drawing showing their original spillway plan and cross-section attached as Figure B.5-8

2. *Repairs of spillway glacis in Spillway bay nos. 3 & 4*

The spillway glacis of bay 3 was repaired using M90 grade concrete with steel fibers and that of bay no. 4 with M90 grade concrete without steel fibers. The minimum thickness of M90 concrete was 500 mm.

For joining the high strength concrete with the original spillway glacis concrete the following procedure adopted:

- Work was taken up from the elevation corresponding to the point of intersection of the spillway piers with the spillway glacis from bottom to top.
- Chipping of spillway glacis to achieve minimum 500 mm thickness for placing rich concrete etc.
- Shear keys of size 1.0 m x 0.3 m with a depth of 0.45 m were cut in alternate shutters at about 5 m c/c.
- Thorough washing of existing concrete was carried out using high pressure air-water jets.
- About 1.5 m long anchors were provided at a spacing of about 2 m c/c at the interface (approximately 1 m deep in existing concrete).
- Bonding agent of BASF used for bonding.
- Concreting done using shutters of size 2.5 m width and 1.25 m height from bottom to top.

The concrete mix design for M90 grade concrete and construction supervision of high strength concrete work carried out by National Council for Cement and Building Materials (NCCBM), Ballabhgarh.

The high strength M90 grade concrete mix design details were as under:

Grade of Concrete:	M90A20 with Chemical Admixture	Exposure Condition:	Severe
Workability:	125-150 mm at 30 minutes	Maximum W/C Ratio:	As per IS: 456
Cement:	OPC 53 Grade (Ultratech)	Minimum Cement Content:	As per IS: 456

Table B.5- 1 Strength grade concrete. (Adapted CWC, 2017)

The recommendations for M90A20 grade of concrete for the target average 28 days compressive strength of 98.25 N/mm^2 below.

Sl. No.	Mix Constituents	For One Cubic Meter of M90 Concrete with Steel Fibers (Kg)	For One Cubic Meter of M90 Concrete without Steel Fibers (Kg)
1.	Cement (OPC 53 Grade)	527	490
2.	Fly Ash	145	135
3.	Silica Fume	55	50
4.	Water	160	152
5.	Fine Aggregate (Crushed)	655	424
6.	Coarse Aggregate 10-20 mm	329 (40 %)	589 (50 %)
7.	Coarse Aggregate < 10 mm	493 (60 %)	589 (50 %)
8.	Chemical Admixture BASF Glenium Sky 8866	12.7 (@ 1.75 % by wt. of cementitious material)	6 (@ 0.9 % by wt. of cementitious material)
9.	Water Cementitious ratio	0.22	0.225
10.	Steel Fibers (1.27 % by volume of concrete)	100	--

Table B.5- 2 Recommendations for M90A20 grade of concrete. (Adapted CWC, 2017)

3. Repairs of deep cavities downstream of the spillway

Three deep scour holes /pits of big size had formed in the bucket area in front of bays 1, 2 & 3. One of the pits in front of bay-3 was the deepest. These pits were reported to be inter-connected. For filling these holes /pits, the following was recommended:

a) First, select the scour hole / pit with the minimum depth for repairs, out of the three for repairs.

b) After dewatering, chipping of the entire concrete surface of the hole /pit to get a rough surface is to be carried out.

c) After cleaning and drying of the pit and provision of dowel bars, fill the pit with M20 concrete with non-shrink admixture by tremie concrete in lifts of 1 m height and giving a time gap of minimum 72 hours between each lift.

d) Provide reinforcement mesh of 20 mm dia. Tor steel bars @ 150 mm c/c both ways in the top layer and flush it with the existing surface. This area is later to be covered with high strength concrete as required for the EDA.

e) Consolidation grouting of the area in the bucket portion to close any inter connections of pits and damaged rock mass with open joints et

4. *Proposed repair of spillway piers*

The d/s vertical face of the piers had also been eroded badly and the reinforcement was exposed. The same recommended repairing as under:

a) To provide cladding with steel plate (20 mm thick) having width equal to the thickness of the spillway piers less about 15 cm on both sides to avoid possible damages to the plate on account of erosion of the pier faces later on. The height of this plate cladding could extend about 1 m higher than the highest damaged point on the pier. The lower elevation of this plate would be the elevation of the point of intersection of the pier with the spillway glacis.

b) This cladding plate to fixed using anchor bolts of 25 mm dia., 1.5 m long at 0.50 m spacing (Staggered) on to the existing concrete face of the pier.

c) The space between the eroded d/s face of the pier and the cladding plate to filled with cement grout / mortar / micro-concrete after tightening the anchor bolts. Also, weld the end of the anchor bolts with the cladding plate.

5. *Construction of proposed stilling basin in place of slotted roller bucket*

In view of problems of rolling boulders and unsymmetrical flows from the spillway it was felt not to reconstruct the slotted roller bucket as it is prone to damages. A stilling basin without appurtenant has been proposed instead. The unsymmetrical flow from the upstream can see from Figure B.5-9.

The spillway originally designed for 5,000 cumec. After review the revised design flood to be adopted for safety of the dam has been determined as 8,368 cumec (SPF). However the maximum flood observed by the project authorities has been 1,457 m^3/s only on 16.06.2013. The dam is likely to get overtopped with the increased flood. This issue is to be looked seriously into by the project authorities.

As regards the energy dissipation arrangements it is considered adequate to design for the original design flood of 5,000 m^3/s only which is about 60 % of the revised flood. The Tail water rating curve was worked out by HEC-RAS model. In addition, it co-related with the original tail water levels available with IRI, Bahadrabad.

As there were, some differences in the tail water levels the hydraulic model studies planned to carry out with both the curves. Also it is planned to consider the upstream reservoir as silted up to the spillway crest for assessing the discharging capacity of the spillway.

The stilling basin design was carried out as per IS 4997using the tail water rating curve obtained by HEC-RAS. A drawing showing the proposed stilling basin attached as Figure B.5-13. A length of 100 m with invert at El. 1258 m has arrived at.

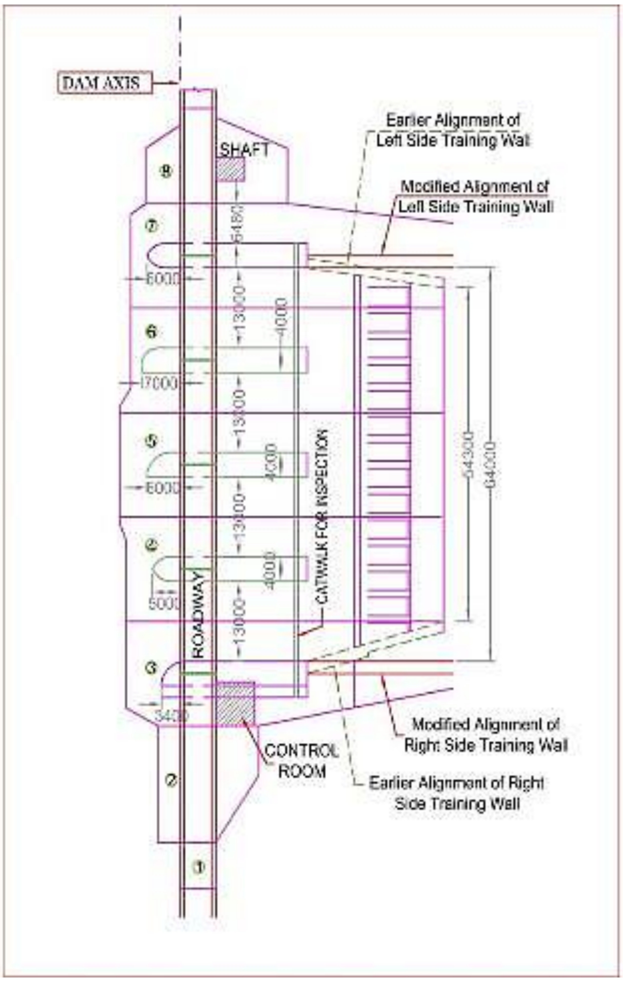
To avoid cutting below the spillway the d/s toe the spillway proposed to extend on the downstream with provision of a small sloping apron before the stilling basin.

The top about 750 mm of the stilling basin proposed to constructed with M90.Presently the hydraulic model studies are under progress at IRI, Bahadrabad. A composite model has been prepared by IRI, Bahadrabad simulating the downstream topography. In order to get required width for the stilling basin in the lower d/s reach the proposal envisages hill cutting on the left bank for extension of training wall.

Slope protection measures will also be required to taken on the left bank. Drawings showing these aspects attached as Figure B.5-10, 11, 12 and 13.

Conclusions

Repair works of spillway glacis in bays no. 1 and 2 carried out by using M60 grade concrete and fixing MS steel plate over prepared profile followed by cement grouting. The repair works of the glacis of spillway bays no 3 & 4 have been carried out under the DRIP using M90 grade concrete for the first time in India both with and without steel fibers. The spillway glacis repairs with M90 concrete have withstood two monsoon seasons successfully. Long term monitoring of the spillway performance with these three diverse types of repair works is to be carried out and conclusions can thereafter be drawn as to which method is the most suitable for repairing of spillway glacis and energy dissipation arrangements for the severe conditions prevalent in this project. The construction of stilling basin and improvement in d/s flow conditions expected to provide sustainable rehabilitation solution for the Maneri dam spillway.



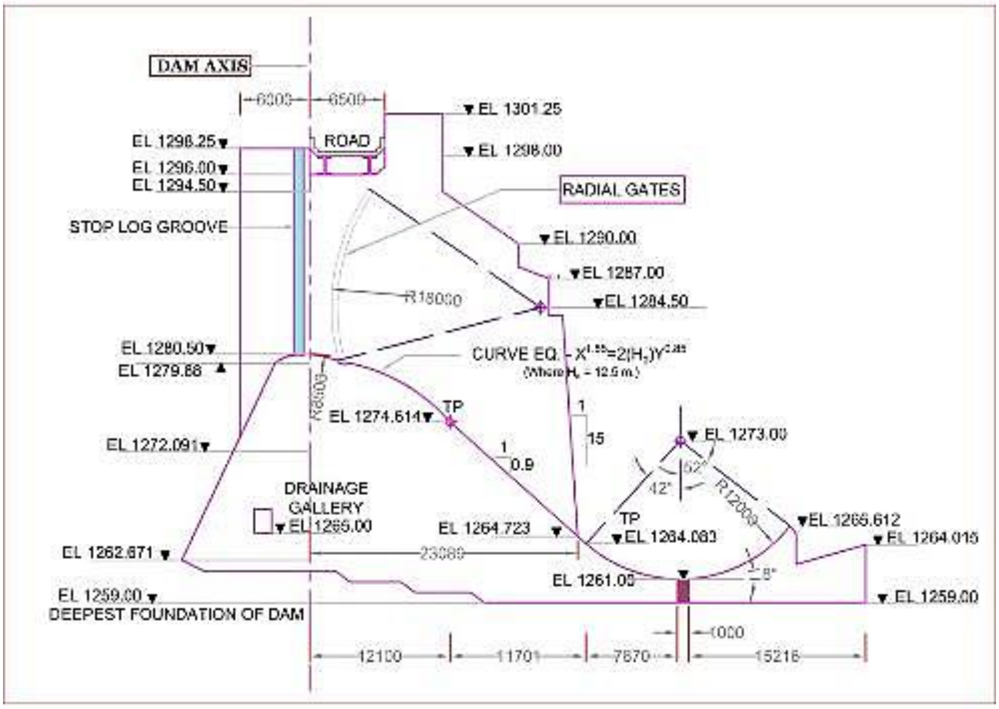
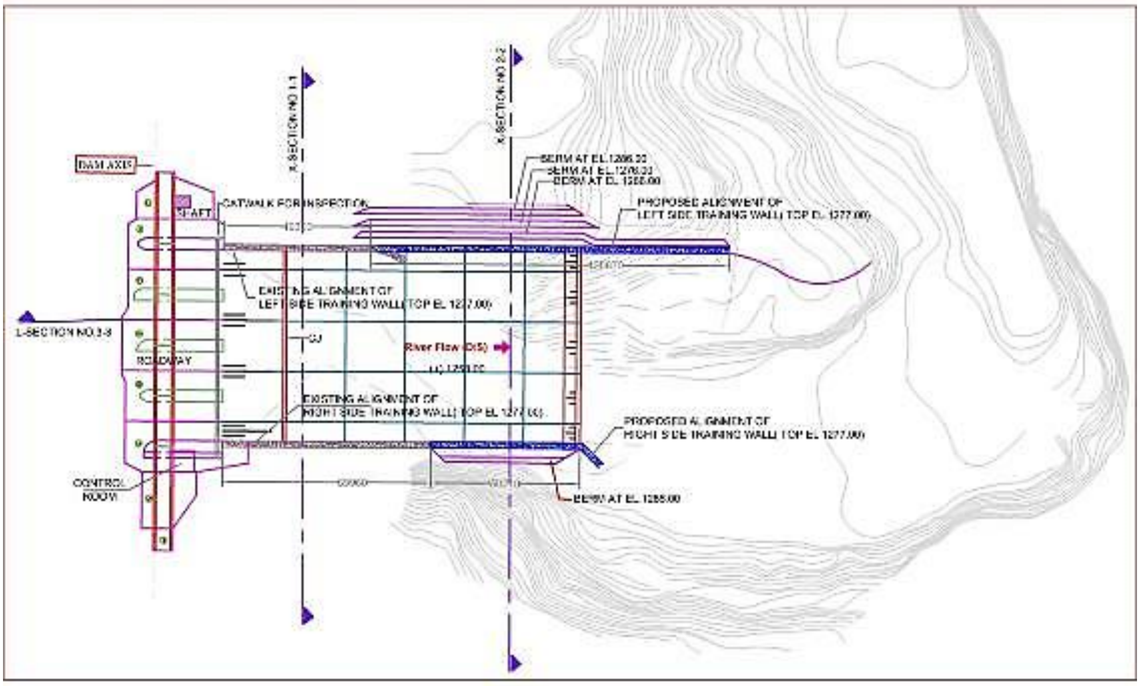


Figure B.5- 8 Original Spillway Plan and Cross Section (CWC)



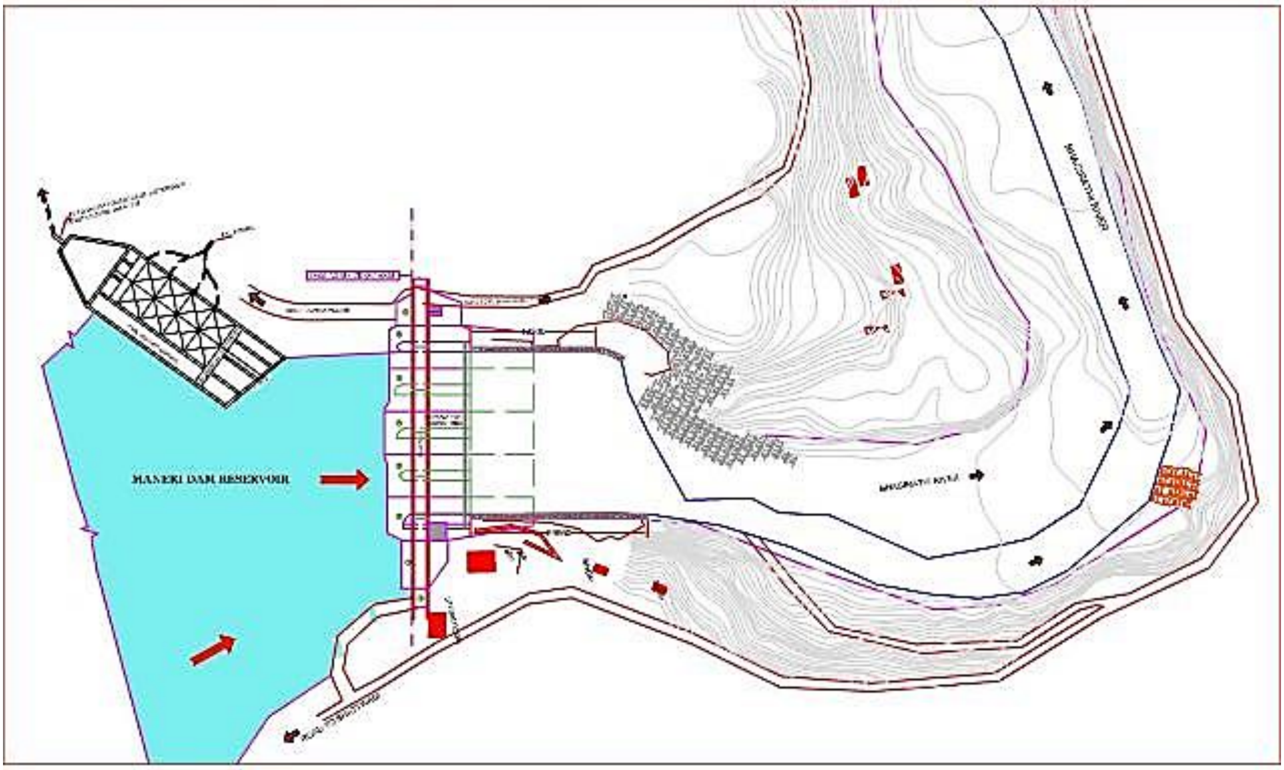


Figure B.5- 9 Unsymmetrical flow from the upstream side (CWC)

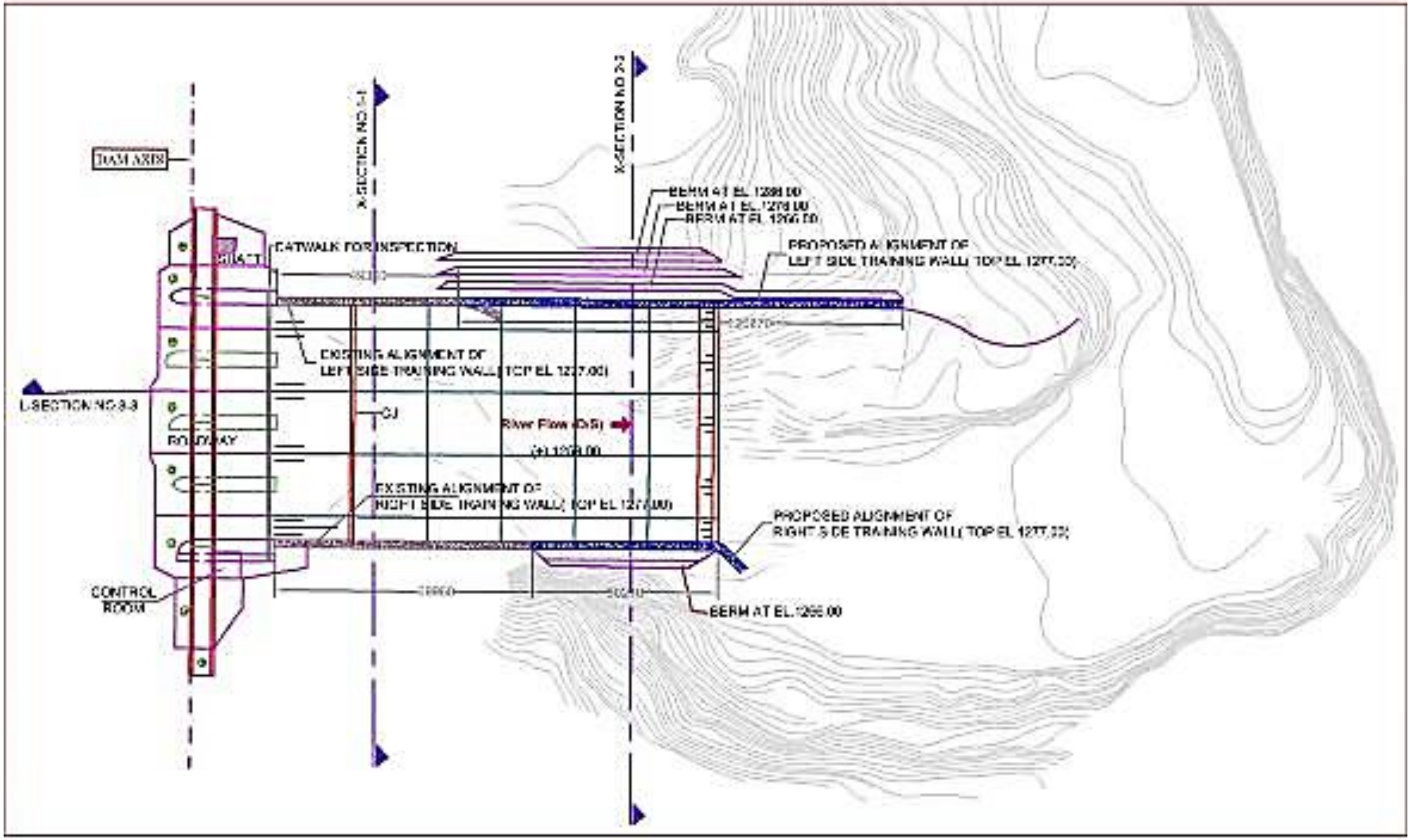


Figure B.5- 10 Layout Plan of Proposed Stilling Basin & Extended Training Walls (CWC)

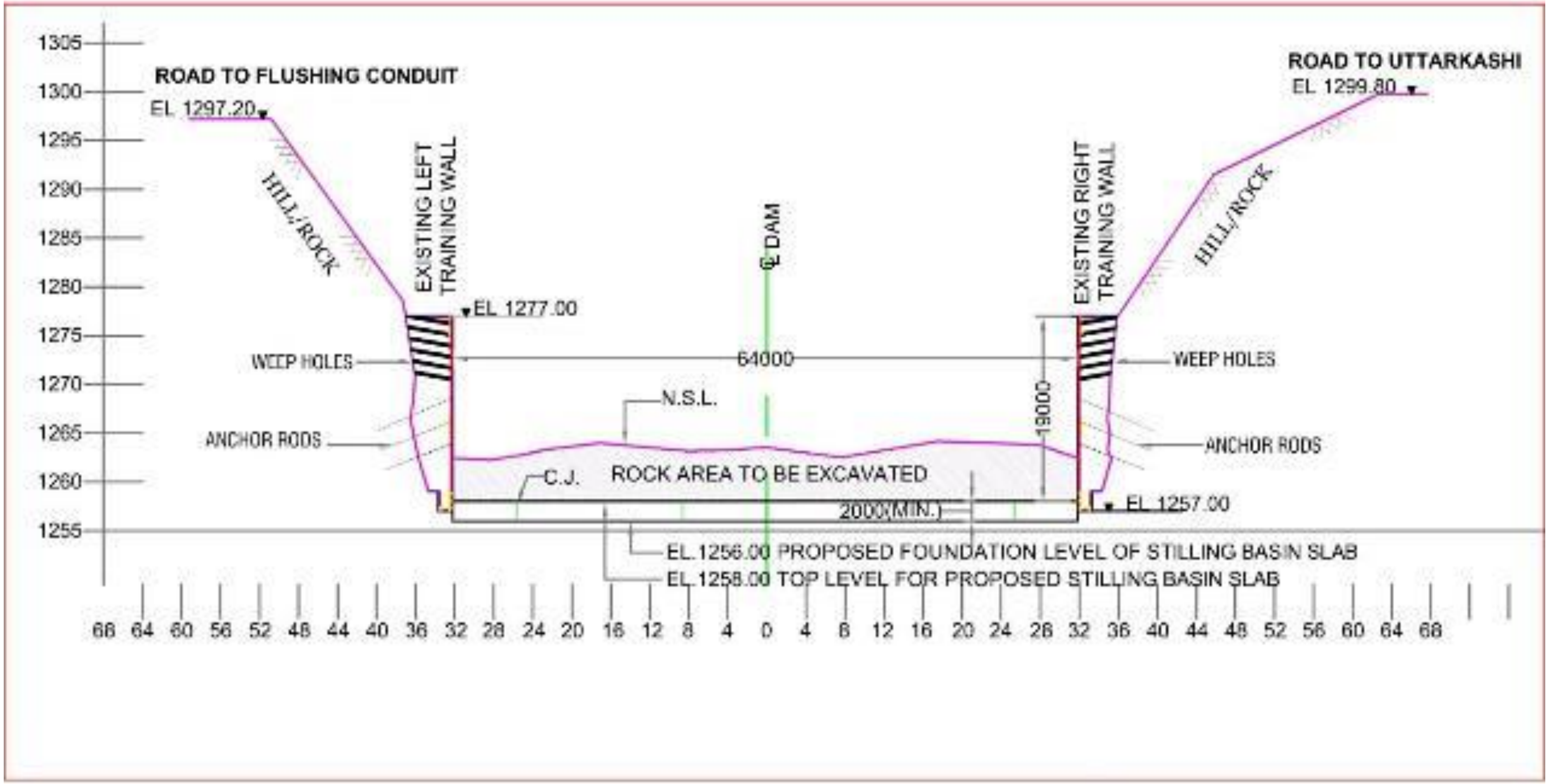


Figure B.5- 11 Cross Section 1-1 at RD 50.00 m from Dam Axis (CWC)

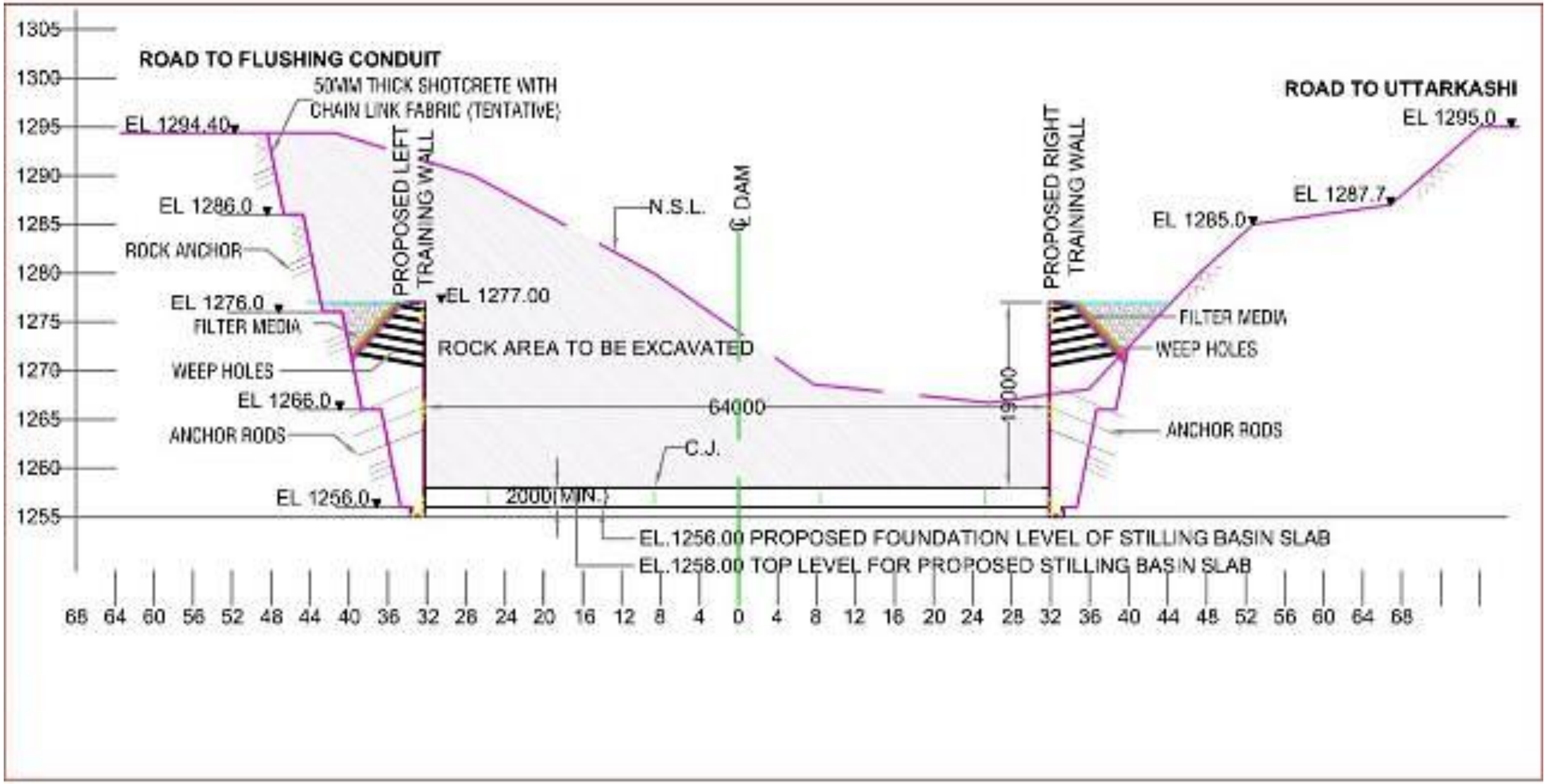


Figure B.5- 12 Cross Section (CWC)

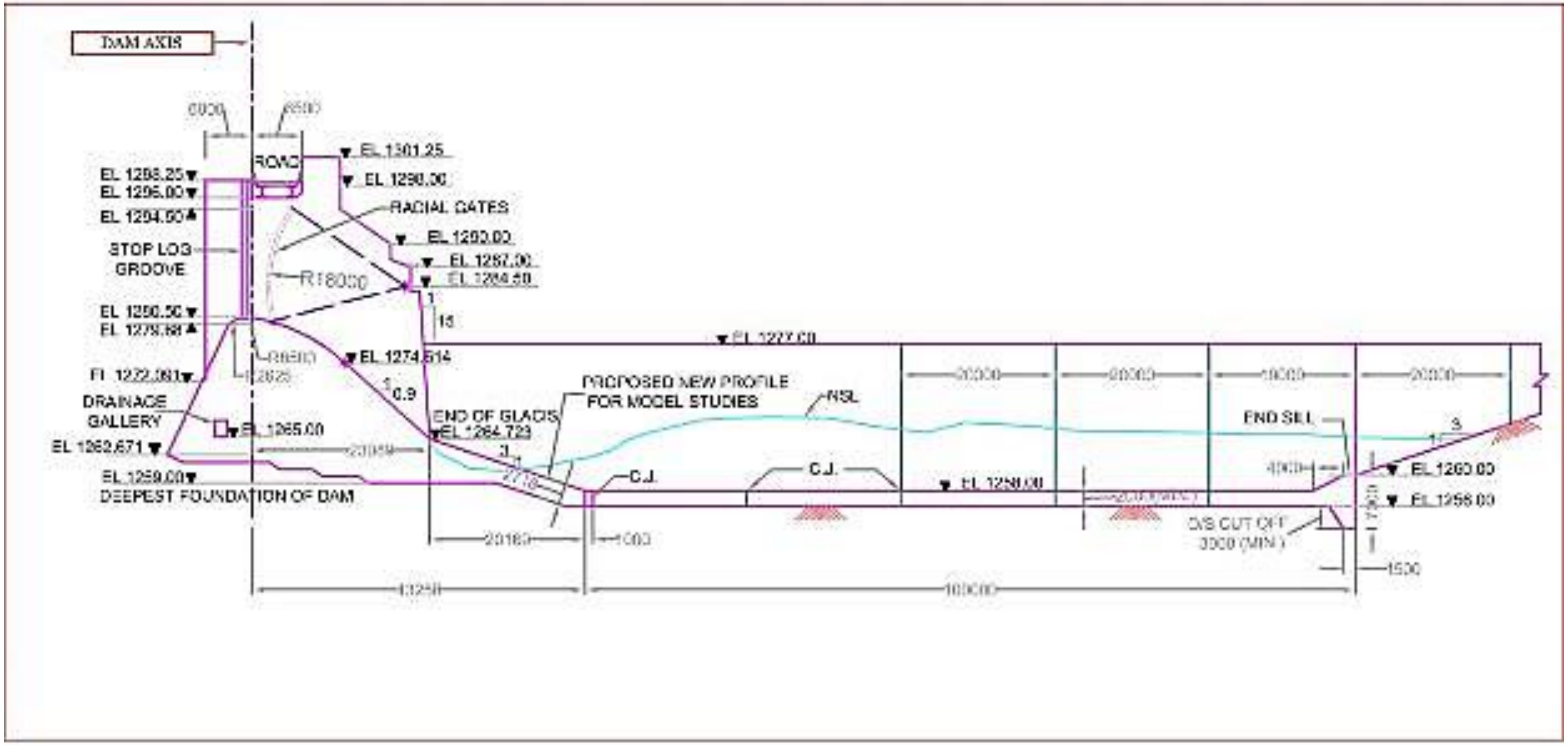


Figure B.5- 13 Longitudinal Section of Proposed Stilling Basin (CWC)

B6-Failure of Main Spillway Radial Gate no.5 at Narayanpur Dam

Abstract

On 6 October 2005, gate number 5 of the Narayanpur dam, located between Bachihal Siddapur in Bijapur district and Hirejavur in Raichur district in the state of Karnataka cracked open with a loud bang around 10:30 AM. Five fishermen who were fishing nearby had a miraculous escape after being swept away for nearly a kilometer by the sudden gush of water. Within minutes, the entire gate ripped out and the stop log gantry crane partially damaged by the torrent of water. No senior official was present at the dam when the accident occurred. Repeated attempts to lower the emergency stop log gate of the damaged gate number 5 failed because of the strong current. Although authorities had opened 13 gates of the dam to bring down the water level and facilitate early lowering of the emergency stop log gate, releases from the upstream Almatti dam continued, and worsened the situation further. Uncontrolled water release from the dam due to this accident was estimated around 540 Mm^3.

Figure B.6- 1 Narayanpur Dam Spillway in Operation. (CWC)

Figure B.6- 2 Stop logs placed in Bay 5 of Narayanpur Spillway after the failure. (CWC)

The project

Narayanpur Dam is built on the Krishna River at Siddapur village in Bijavour District, Karnakata State in Southwestern India (see Figure B.6-1). Dam type is composite of earth and masonry gravity dam. Dam length is approximately 8.5 Km with maximum height of 25.8m. Its purposes are irrigation, hydropower generation and domestic water supply. Its construction started in 1964 and finished in 1982. The reservoir that it impounds is known as Basara Sagar and has 863 Mm^3 of live capacity and 1,066 Mm^3 gross storage. The spillway is of the Ogee type controlled by 30 radial gates of size 15m x 12m provided with stop logs, 3 sets of 10 elements of size 1.5m x 15m.

Brief Description of Spillway Radial gates and Hoists.

The spillway Radial Gates of Narayanpur Dam are 30 Nos. having size of 15m x 12m used to regulate the design flood discharge of about 45,000 m^3/s in the reservoir corresponding to MWL/FRL of 492.252m. Each gate is operated independently by a Rope Drum Hoist mounted on hoist chassis. The hoists are having Rope Drums over which wire ropes wound and connected to the gates on the upstream side of skin plate. The design of the Gate parts and hoists confirm to IS 4623–1967. The Gates provided with side and bottom rubber seals to prevent leakage**.**

Technical data:

1. No. of Gates	: 30 Nos
2. Size of Gates	: 15 m x 12 m
3. Radius of Gates	: 12.00 m
4. Top of Gate	: 492.650 masl
5. Maximum Water Level	: 492.250 masl
6. Full Reservoir Level	: 492.250 masl
7. Clear width of opening	: 15 m
8. Elevation of Spillway Crest	: 480.250 masl
9. Elevation of sill	: 480.100 masl
10. Elevation of Trunnion pin centre	: 484.650 masl

Hoist details:

1. Capacity of Hoist	: 100 M. T
2. Type of hoist	: Rope Drum Hoist
3. Lifting/ Lowering speed of the Gate	: 0.426 m/Minutes
4. Braking device	: Electro-magnetic brake 250mm dia 440V AC Single phase.
5. Maximum Lift of Gate	: 10.50m.
6. Hoisting Rope	: 6x37 Const. Hemp main core 40mm dia
7. Drive	: Squirrel cage induction motor 15Hp. 960 RPM 440/400V. Cycles power factor 0.8
8. Control	: Reversible type push button starter Located in the Driver unit.

Sequence of Failure:

Spillway radial gate at bay No. 5 failed due to the failure of the joint between the Yoke girder and the trunnion bracket base plate (Figure B.6-3). On 6 October 2005, the reservoir elevation was at FRL 492.252 masl and the central spillway gates in open position releasing excess flood. Failure of welded joints resulted in huge strain/ distress at the junction of arms and trunnion with the entire load of the gate coming on the left arm assembly, which lead to breaking of the bolts connecting the trunnion to the rest beam. In the absence of support on the downstream side, skin plate assembly along with the horizontal girder was held only by wire ropes on the up-stream side. Wire ropes could not hold the gate in position against the huge water load and eventually snapped its connection with the skin plate. The entire skin plate assembly with the horizontal girders, left side arm assembly with trunnion slide along the spillway profile. Due to high velocity flow, the gate pushed out of the spillway bucket and moved nearly 35 meter into the river. The right arm assembly with the trunnion also slid along the spillway profile. The right arm assembly with the trunnion was also washed away by the high velocity flow into the riverbed and found near the skin plate assembly (see Figure B.6-3 and B.6-4).

Subsequent inspection was carried out after lowering of reservoir level to El. 486.5 masl and thereafter 7 stop log elements were attempted to be placed for lowering the flowing water in bay 5 and finally, the flow was stopped after placing the stop logs with sustained efforts (see Figure B.6-2).

Figure B.6- 3 Typical tie beam joint. (CWC)

Figure B.6- 4 Spillway gate 5 tie beam left joint (weld joint not before failure to the designed penetration thickness). (CWC)

Figure B.6- 5 Damaged Skin plate of gate 5. (CWC)

Causes of Failure

After the detailed inspection, the following causes of the incident were identified:

- Inadequate / Improper weld Joints (filler joint) at the junction of the tie beam and trunnion. Welding did not properly penetrate up to the trunnion and rods and plates were found in weld gaps (Figure B.6-7).
- Formation of scale / corrosion at steel / load bearing surfaces due to the ingress of moisture and water. Weld failure due to progressive rusting along the inside surface of the weld over a period of time.
- Bottom portion of the pedestal lifted up to 10 mm and gap existed between rest beam and pedestal.

- Erection defects like non- matching of skin plate and wall plate, improper fixing of wall plate to the designed vent width and, loss of curvature of skin plate during erection
- Inadequate Inspection and maintenance (limited lubrication, replacement of wire ropes and seals and periodic painting in the successive years).

Figure B.6- 6 Spillway gate No 5 - Tie beam right side weld joint. (CWC)

Remedial Action under taken:

The following actions were performed to rehabilitate the gate:

a) All weld joints redone to the designed thickness.
b) Scale formations on skin plates removed and painted.
c) Stiffeners, which have lost thickness by more than 2 mm, replaced.
d) Strengthen the joints of trunnion tie and the joint between tie beam and the trunnions.
e) The damaged radial gate replaced with a new radial gate in April- 2007.
f) Other recommendations of the committee were implemented in a phased manner from 2007 to 2011.
g) Provision were made Zinc Metalizing to the corroded and gate components such as skin plate, stiffeners, vertical girders and horizontal girders.
h) Provision of a new 84-ton capacity Gantry crane and one set of stop log gates (under DRIP).
I) Repair of all spillway radial gates was included in DRIP with other hydro mechanical works for a comprehensive intervention /rehabilitation. Zinc Metalizing was applied to the corroded and gate components such as skin plate, stiffeners, vertical girders and horizontal girders, Installation of the new 84 Ton capacity gantry crane with lifting beam and one set of stop logs have been carried out under DRIP.

Lessons learned from the incident:

- Joints must be designed for the subjected load with adequate/ standard safety margin and properly detailed in the drawings.
- Fabrication / manufacturing and erection of components should carry out as per approved drawings, approved quality assurance plan and applicable standard / norms.
- Proper Inspection should be carried out at all stages i.e. fabrication / manufacturing, installation and regularly /periodically after installation and commissioning of gate / equipment. Remedial measures based on inspections findings, should take up regularly.
- Critical examination of structural deficiencies e.g. rusting / corrosion and crack in surfaces / weld joints is essential even during operation (after commissioning) and modern techniques such as thermal imaging and ultra sound / MPT/ DPT for such detection can be used.
- Provision of Bulkhead gate with their operating equipment capable of lowering the gate in flowing water condition may be kept to be used during such emergency or on failure of any gate.
- Stocking of necessary spares parts for the hoes, rope, rubber seals, bearings, guides/guide wheels etc. and tools and tackles along with necessary machinery /equipment for welding / cutting and handling e.g. wire rope , clamp , chain pulley block of adequate capacity etc. shall be readily available at site and in good working conditions

APPENDIX C–RETENTION OF DEBRIS BY MEANS OF FLOATING BARRIERS (BOOMS)

Overview

Booms are designed not only for retaining debris, also they act as a device for public safety such as satisfactory boat restraining barrier. The effectiveness of each installation should be evaluated on case-by-case basis.

Barriers general guidelines:

- It is important to identify the best location of the barrier for each dam.
- Barriers should be in narrow locations for costs reasons, they may also be away from strong currents, say some distance away from intakes and/or spillways watercourses.
- It is advised to test the barriers when possible when prototype observations, opening the spillway gates to ensure they will be operative for floods conditions.
- Effectiveness of barriers can be improved by making the barrier more visible. That detail may also help to spot the debris accumulation at places.
- Tuff booms may work well for low flow velocities where they are a useful tool for both retaining and guiding wood logs.
- For high velocity flood situations with large volume of log wood, robustness is not guaranteed, since many failures are reported.
- They can be equipped with underwater net to further reduce passing of wood logs, and fresh wood logs.
- They serve to stop floating logs to pass and/or redirect debris to sector of the reservoir where they are away from critical areas.
- Some authors find them useful only in limited cases when logs are small or medium size. They find them not suitable for large logs.
- Forces on the barrier are considerable.
- Barriers should be removed in winter due to deterioration by ice. Consider flexibility of the barrier in reservoirs with water elevation changes.
- A well designed floating barrier should, as a principle, handle all types of debris considered to appear in the reservoir.

Types of debris

Different types of material have been described previously as floating debris including logs, vegetative and artificial man-made materials (C-1 to C-10). In general practice, booms should also be provided with capacity to retain boats of small proportions.

Figure C- 1 Log wood and other floating debris deposited in front of an intake.

Figure C- 2 Accumulation of free floating macrophytes. (Eichhorniacrassipes or Water Hyacinth due to lack of retention, Flood protection System of the Orinoco River Delta, Venezuela)

Figure C- 3 Process of flushing of Water Hyacinth by gate operation (Orinoco River Delta, Venezuela)

Vegetative debris include certain species highly invasive, owing to characteristics such as high growth rates and the formation of dense floating mats that drift on wind and water currents. Water hyacinth (Eichhorniacrassipes) is one example: its invasion of tropical and subtropical freshwater systems worldwide harms.

Figure C- 4 Man-made floating debris in a reservoir in an urban area, Macagua Reservoir, Caroni River, Venezuela. Featuring loose elements of Boom failed by broken wire (Taken by the author)

Figure C- 5 Ice is a common type of floating debris in Northern Latitudes.

Figure C- 6 Satellite view of Lake Erie-Niagara showing Ice Booms (March 2021)

Figure C- 7 Accident with barge loaded with a truck in a dam forebay,
Tocoma Project, Venezuela.

Figure C- 8 Precarious Mining Barge creating an emergency in a dam without retention system, Caruachi Project, Venezuela.

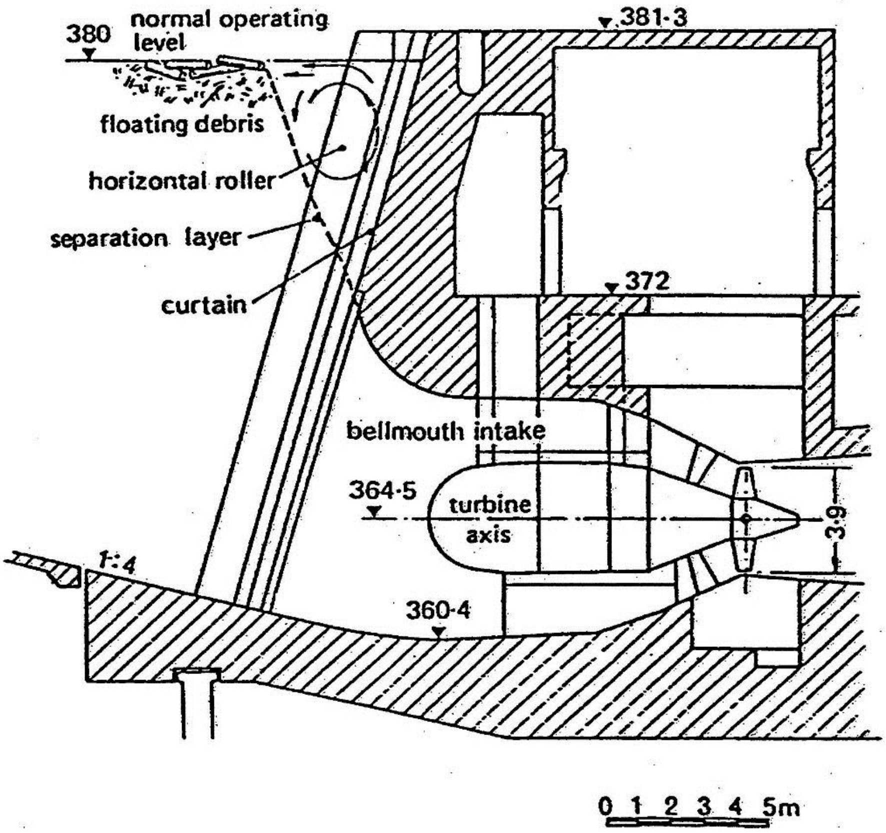


Figure C- 9 Run-of river turbine with separation layer and current less zone showing floating debris accumulation area (longitudinal section) (Wallesrtein et al. 1997)

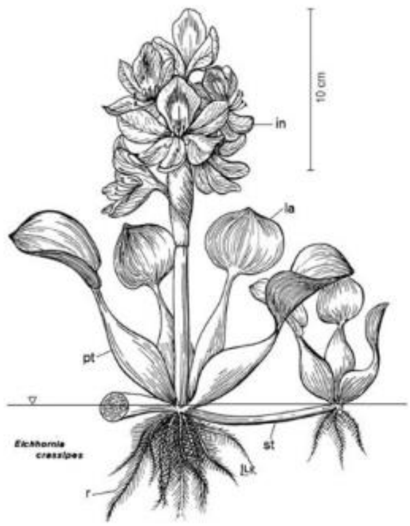


Figure C- 10 Vegetative structure of Eichhorniacrassipes (Downing Kuntz, 2011)

Major mechanisms by which debris enters water courses.

It is important to mention the major mechanisms by which debris enters water courses:

a)	Wind and wave action	Structures such as docks can be damaged by waves, and much of the debris generated remains in the water. On lakes and large rivers waves erode the shoreline causing trees to topple into the water. Wind throw is a major source of debris input in streams in forested areas.
b)	Ice Break-up	Ice storms can cause tree limbs and sections of trunks to break off and fall into lakes and watercourses. Moving ice in the spring break-up can increase the undercutting of riverbanks, and trees can be damaged and broken by the force of moving ice.
c)	Forest Litter	A larger litter input is derived from leaves from deciduous trees and some conifers. Forest litter is usually protected by the tree canopy during summer and by a snow layer in the winter, however in early spring trees are without leaves and heavy rains will wash the litter into watercourses.
d)	Forestry Practices	Forest lands soak up large quantities of water and reduce floods and erosion that bring floating debris to the streams and rivers. If a generous ground cover is maintained during tree harvest and roads are made erosion resistant, forest land can still protect the watershed. The harvest of trees on a reasonable schedule will reduce the number of dead trees that may fall into the streams and rivers. However, poor harvesting strategies can generate large inputs of debris to streams and rivers.
e)	Debris Jams	Debris jams may release debris downstream when moved en-mass by a large flood flow or when broken down over a long period of time by natural effects such as decomposition.
f)	Beaver Dams	The quantity of debris brought into streams by beavers is unknown but may be a substantial proportion of the total debris load in some watersheds.
g)	Floating Vegetation	Floating vegetation may grow very fast and create an almost solid surface on the water, that can exert substantial forces on structures and generate hassles to nearby population as high mosquito concentration and anacondas grow site, if they are left to grow without removal. Usually, it is present in tropical regions. Its growth is also present in reservoir areas with cities nearby.
h)	Man-made Materials	This includes decaying wooden structures such as piers and wharves, and organic and synthetic material from dumps improperly located along water bodies, and general littering of trash and waste.

Table C- 1. Debris major mechanisms for entering in water courses (adapted from Wallerstein et al, 1997)

Site investigations

In general, floating barriers should be designed to support all kind of Floating Debris which eventually can accumulate in front of the structures, the intake, or the spillway. These includes debris (wood logs, vegetation, etc.), ice and eventually should be capable of retaining small boats.

Design of a Floating barrier include to investigate many aspects such as river basin log production, types of debris, chain stability and attachment to the shore, wear of temporary elements either floating and metallic, type of wood, changing of wood buoyancy due to wood saturation, detention capacity of water by the wood log, fluctuation of water levels in the reservoir.

Site Investigations may include survey, field data taken on the following:

- River basin estimation of vegetation's, log characteristics, log wood production, use of land
- Bathymetric survey of the reservoir
- Currents regime in the reservoir, at selected alternatives of locations for the barrier
- Reservoir Flow Velocity pattern for normal and flood sceneries
- Presence of Ice and characterization of different types of floating debris
- Wind and wind induced waves
- Fluctuating range of reservoir levels

Computer simulation (Valbuena et alia, 2010) will aid in getting to know the best places for the barrier to be installed according to the reservoir bed topography and flow velocity patterns. GPS and ADCP (Acoustic Doppler Current Profiler) can provide with the required bathymetry and water flow velocities in existing reservoirs. 2D and 3D numerical models may aid to develop reservoir currents in flood scenarios assessing in the design and location of the boom barrier.

Site selection for the Floating Barrier

For very cold regions, the site is usually selected to ensure the ice will form and progress upstream of the boom as soon as possible. The ice cover will provide a storage area, for the frazil and slush to be deposited under the cover, rather than drifting to the power plant intakes or to the spillway structure. The presence of an ice cover will provide an area for the cooled water to warm down before it reaches the power plant water intakes (Abdelnour, 2003).

Bathymetric survey

The reservoir bathymetry provides an important input for the selection of the site. Bed features may determine the length of the cables for anchoring at the different stations and difficulty of placing the dead blocks in reservoirs already formed. Also, it will permit to define boat and users passes at some locations of the barrier.

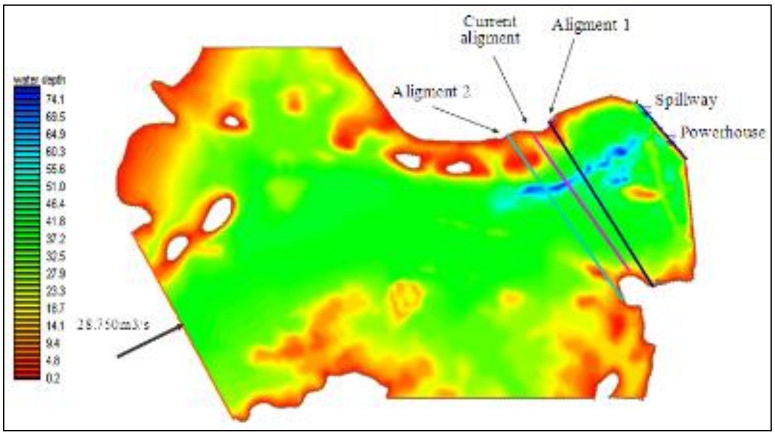


Figure C- 11 Water depth in the reservoir at selected alignments of Booms Barrier

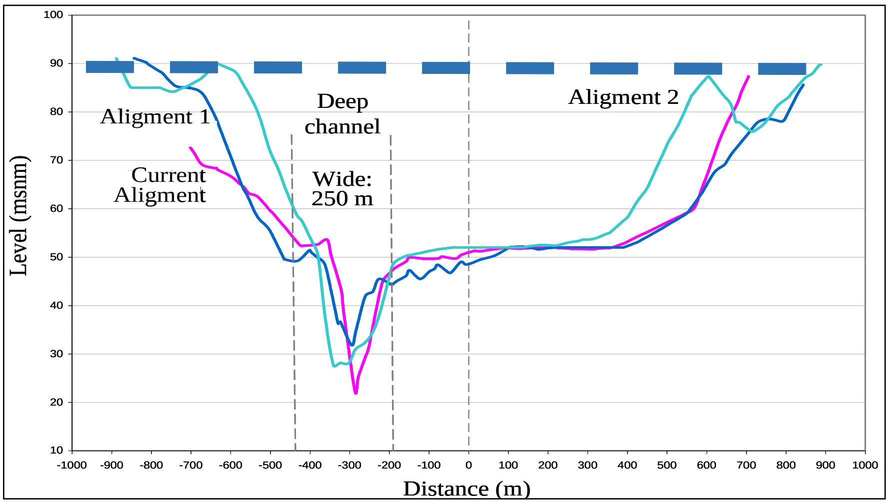


Figure C- 12 Bathymetric Profile of the Alignments (Valbuena et alia, 2010)

Reservoir Flow Velocity pattern

Flow velocity also plays a fundamental role in the location of the barrier. It is true that barrier should be located as near as possible to the intake or water spilling structure, but large velocities occasioned by near places of high gradient, will tend to develop large forces and malfunctioning of the barrier particularly during large floods.

Investigations to optimize the alignment of the barrier may include 2D reservoir simulation and Prototype velocity measurements (Figure C-11 to Figure C-15), Valbuena et alia, 2010).

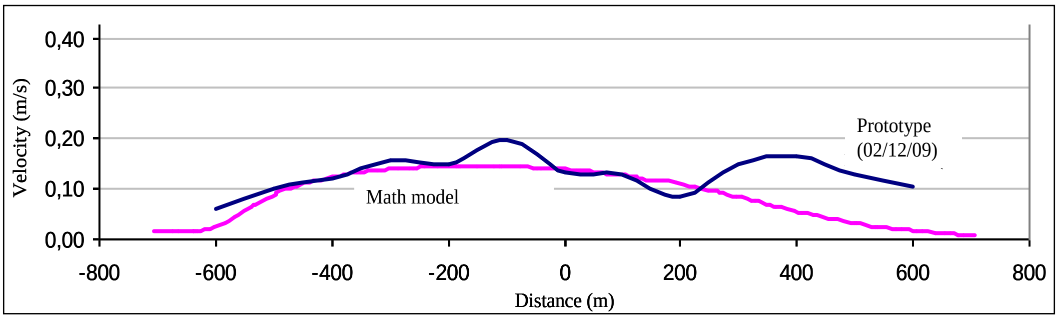


Figure C- 13 Correlation prototype-Numerical Model for a given Boom alignment (Valbuena, et alia, 2010)

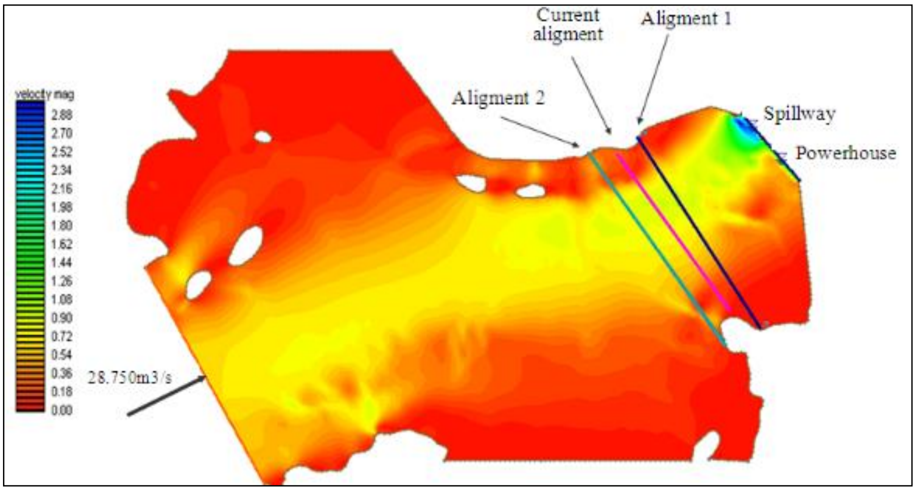


Figure C- 14 General flow velocity pattern in the Booms area and location of selected alignment locations (Valbuena et alia, 2010)

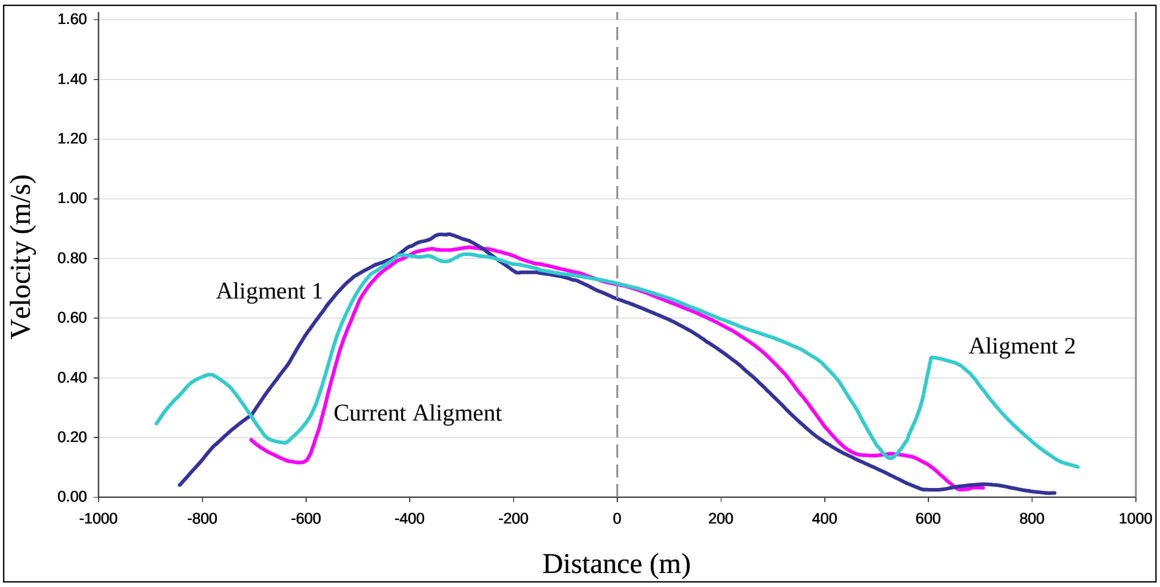


Figure C- 15 Velocities along the barrier alignment (Valbuena et alia, 2010)

Floating Barrier Components

Floating Barrier System has the following elements. (see Figure C-16)

1. Floating Booms
2. Anchors to dry shores of the reservoir
3. Anchors to the bottom of the reservoir
4. Wire Ropes (cables) and steel chain elements that put all elements together.

The final purpose of the design is to select the proper Wire Rope Class that can withstand the forces of the expected floating debris and wind forces acting on the boom which will be transmitted by the wire ropes to the anchors at the bottom of the reservoir).

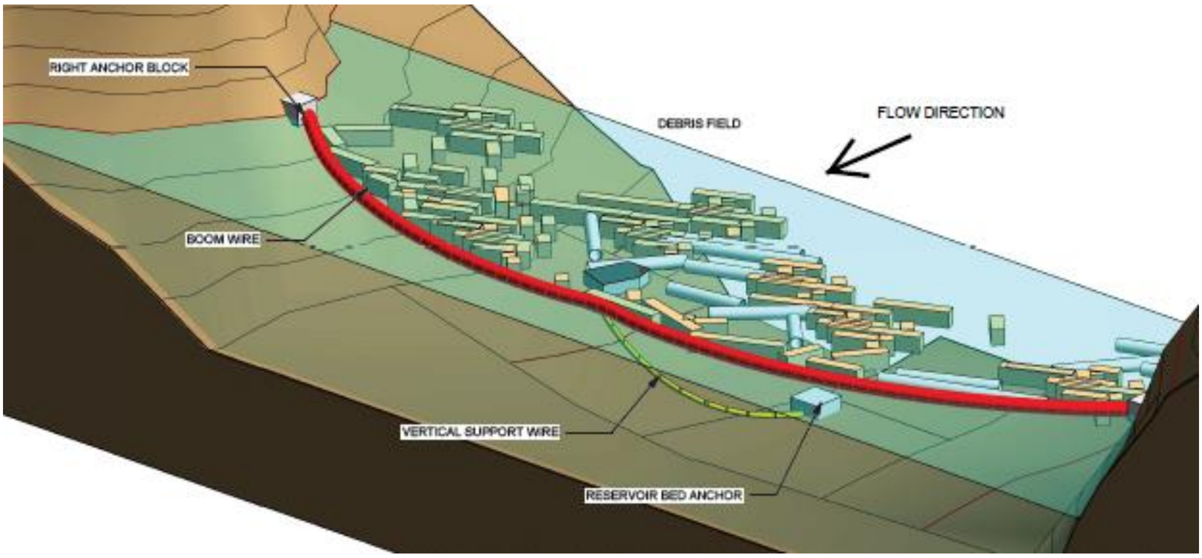


Figure C- 16 Isometric View of a Floating Barrier System showing its main components and a floating debris field.

Floating Catenary

Floating Debris Drag and wind forces act on all floating booms, forming a floating catenary (C-17) with a projection under the water surface enough to prevent the flow of floating debris of entering other areas of the reservoir.

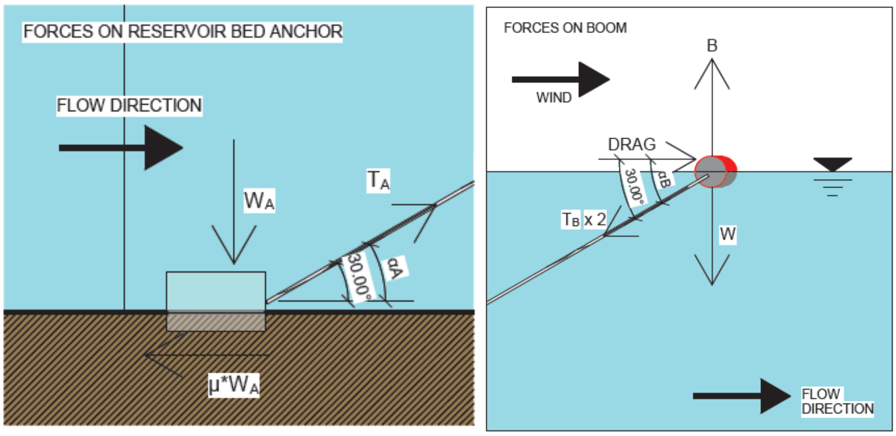


Figure C- 17 Forces acting on the Floating Barrier. Wire ropes transmit the loads in the reservoir surface exerted by the floating debris and the wind, to the anchors at the bottom of the reservoir.

Submerged Catenary

When the floating catenary is very long, submerged anchors should be used. The wire rope used to contain the floating catenary, will create a submerged catenary (Figure C-18) due to its own weight. This tension from the submerged catenary will be transmitted to the anchors, piers or other method used to fix the floating barrier to the bottom of the reservoir by the submerged wires (Figure C-18).

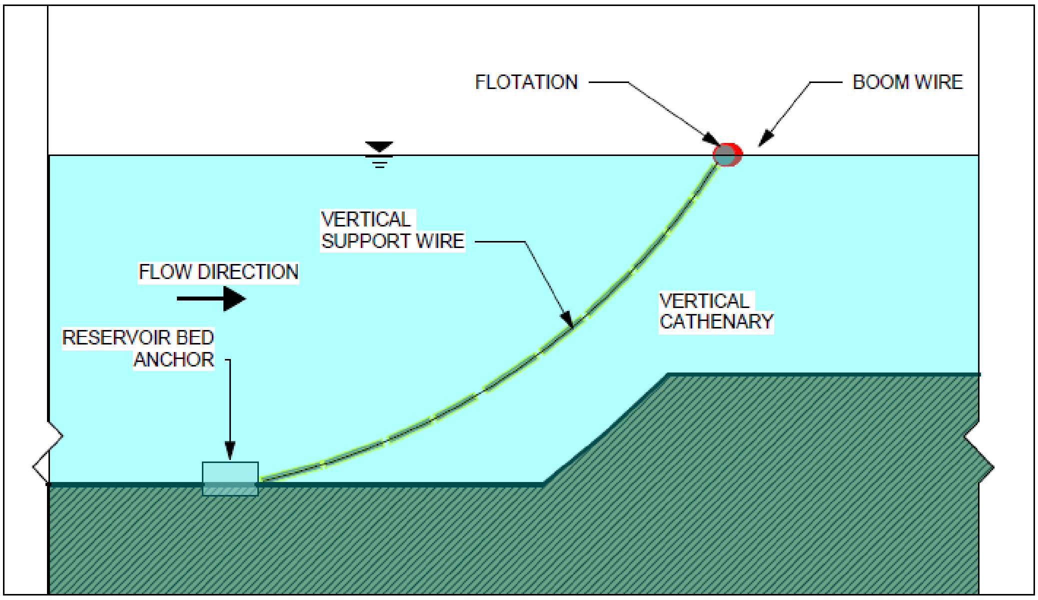


Figure C- 18 Longitudinal View showing the submerged catenary formed by the weight of the wire rope anchoring the boom to the bottom of the reservoir.

Floating Debris Forces on the Floating Barrier - affecting factors.

Floating debris force on the floating barrier is the result of the drag force of the floating debris tangle acting produced by the drag generated by the relative velocity between the water at the reservoir surface and the debris tangle. The barrier will develop debris accumulation (Figure C-19) on the reservoir free surface that will generate a load on the floating barrier. This force is proportional to the area of the accumulated debris (Wallerstein et al, 1997). If the accumulation area of debris is larger than the debris accumulation design area, then the barrier may fail by breakage of the wire ropes or by overtopping.

Tests have given a general formula for calculation of the flow forces (F_W) on anchored trash (Wallerstein et al, 1997):

$$F_w = C_d b(30t + L)\frac{\rho v^2}{2}$$

where:
C_d is the drag coeficient
C_d= 0.06 for $v < v_{su}$;
C_d= 0.08 for $vs< v <1.1 \ v_{su}$;
C_d= 0.10 for $v > 1.1 \ v_{su}$;
ρ = Density of water, kg/m³
v = Water surface velocity, m/s
v_{su} = Flow velocity underneath the debris jam, m/s
b = Width of obstracting debris normal to flow, m
L = Length of obstracting debris in flow direction, m
t = Height of obstracting debris, m

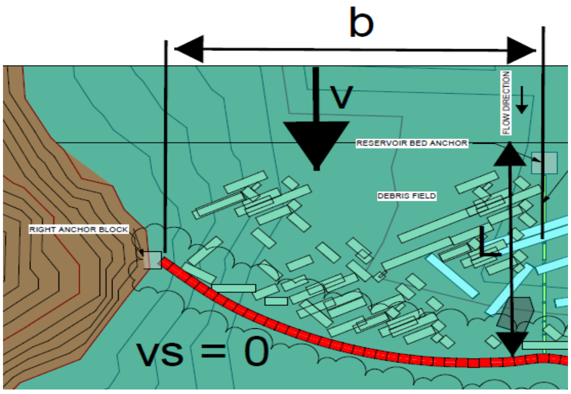


Figure C- 19 Factors affecting the forces exerted by the debris.

Water surface in contact with debris creates a shear plane responsible of the shear drag force. Also, the floating debris depth has an influence in the force against the floating barrier.

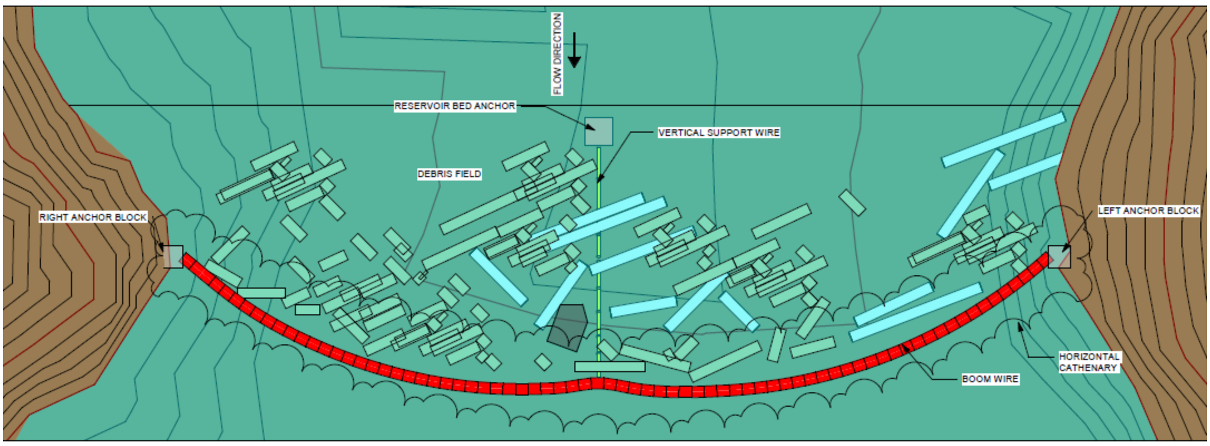


Figure C- 20 Plan View of a Floating Barrier System showing the catenaries formed by the distributed forces exerted by the floating debris accumulation.

Wind and waves normally contribute little to the total anchor force, unless the wind and consequent waves are of extraordinary strength.

Wind Forces on the Floating Barrier

Wind may generate a very strong effect on the barrier, particularly when severe tropical storms and tropical cyclone, occur on the reservoir location. For the design of the barrier, it is supposed the worst-case scenario corresponds to that when the wind generated by the storm is in the same direction of the water flow.

There are five divisions or Regional Bodies that form the World Meteorological Organization (WMO), shown in Figure C-21. Each Regional Bodies oversees radar and satellite observations, warnings coordination, aircraft reconnaissance (where applicable) of Tropical Cyclones that occur in the associated land masses and their surrounding islands and oceanic areas.

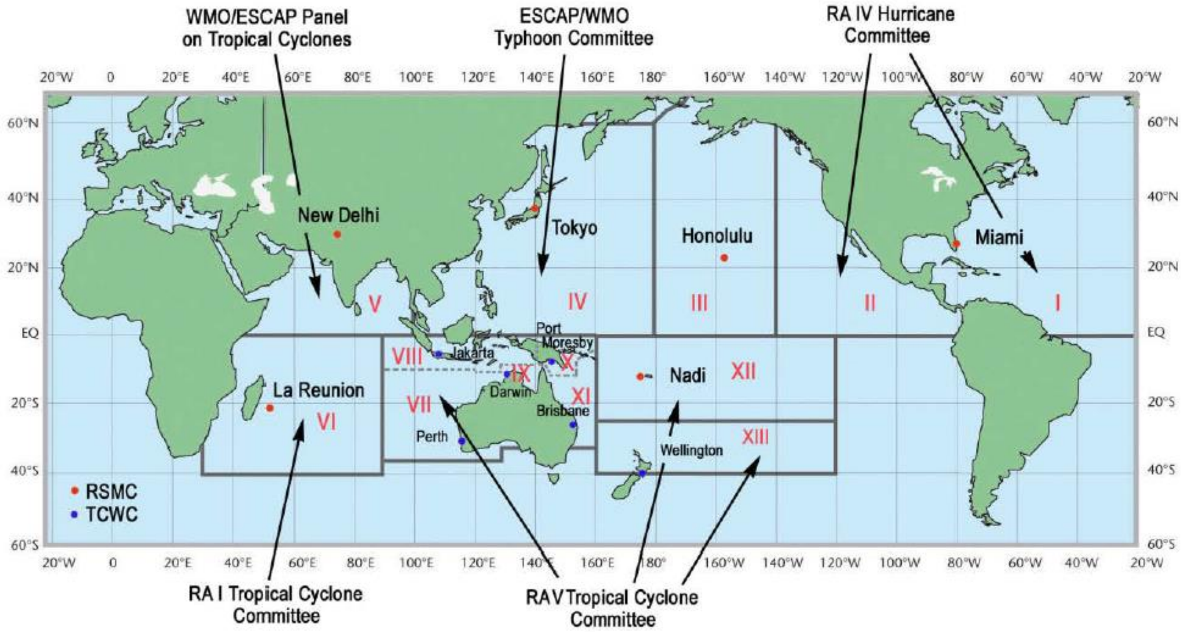


Figure C- 21 World Meteorological Association (WMO) map of the five Tropical Cyclone (TC) Regional Bodies associated to the geographical regions of TC occurrence.

Figure C-22 shows a comparison of the different terminology of tropical cyclones used worldwide by the Regional Bodies. Tropical cyclone wind velocities can be very high with values up to 250-260 km/h, for very intense tropical cyclone.

Calculation shows that wind loads can be significant and have the same order of magnitude of the forces exerted by the floating debris, because wind drag on the floating barrier follows classic drag model which establishes that the drag force is proportional to the square of the wind velocity.

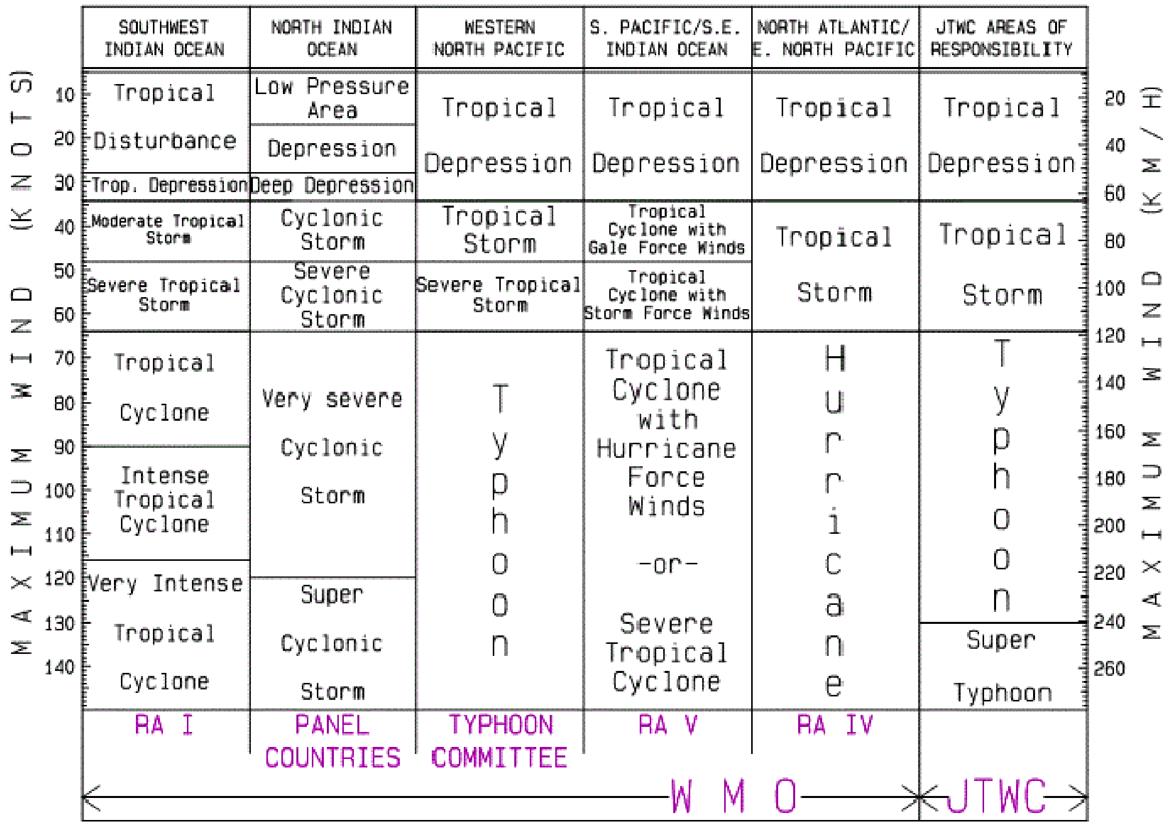


Figure C- 22 Tropical Cyclones terminology comparison table, according to World Meteorology Organization (WMO) and Joint Typhoon Warning Center (JTWC) showing wind speed ranges.

Floating Barrier Design Methodology

The design process of a Floating Barrier System is very site dependent because every site has specific condition that must be evaluated prior to make a design.

The minimal input required from any reservoir are the following:

1. The maximum water velocity in the reservoir,
2. The maximum wind velocity expected according to weather historical record,
3. Knowledge of the possible floating vegetation, floating debris, ice formation, etc.,
4. The depth range in the area where the boom system and the anchors to the bottom of the reservoir will be placed (reservoir depth), and
5. The reservoir level variation may have an impact on the anchors positioning.

All these parameters will be used to determine the forces on the components and more importantly on the wire ropes which keep the system components together, based on the anchors location and reservoir depth at those points and the shore-to-shore distance in the selected points for floating barrier location in the reservoir.

This page has been left blank intentionally.

APPENDIX D–HYDRAULIC MODELING

In hydraulic engineering, physical and computational models have been applied to address many concerns which include: hydraulic structures and waterways design/operation; effects of navigation traffic on waterways; water and environmental quality issues; contamination of groundwater resources; wetland management; flood-control; channel design and operation; control of erosion; and watershed runoff and flow analysis, etc. Recent development in numerical simulation models include advancement of computational techniques and understanding of physical process within the models, and improved modeling approach through the coupling of models with graphic interface, visualization, and parameter estimation methods. Linkages to Geographic System (GIS), optimization and uncertainty evaluation methods, and decision support techniques are also key elements of these system developments. The new generation of modeling systems is providing increased capability to hydraulic modeling specialists and better insights of the dam hydraulics to the managers and decision-makers.

This Appendix is divided in a total of 18 Sections and attempt to cover the topic of Assessing Hydraulic Modelling for Safety of Dams:

1. Hydraulic Model Studies for Dam Safety Assessment Considering Climate Change

Potential extreme weather condition related to climate change should be seriously taken into account in performing the dam safety assessment and the related hydraulic model studies.

On August 7th, 2010, the region of Niedów, Radomierzyce and Bogatynia in Poland experienced a sudden and extremely high rainfall. The capacity of the Niedów reservoir, which is located on the Witka River, was exceeded and caused overtopping and failure of the two dykes of the reservoir. The flood discharge of that year was three times larger than the highest recorded flood on the Witka River. After the catastrophe, the authorities immediately ordered the reconstruction of the Niedów barrage. The proposed modernization/rehabilitation scheme consists of substituting part of the right dike with a labyrinth weir and replacing the rest of the previous earth fill dyke with a concrete buttress wall. Both physical and numerical hydraulic studies were carried out to finalize the design. Through this experience, it was demonstrated that the combination of numerical and physical hydraulic modeling approach represents one of the most promising tools of the future for analyzing climate change related extreme conditions [97].

In the recent declaration by the International Commission of Large Dams (ICOLD) entitled "*World declaration on Dam Safety*", it is indicated that one of the primary benefits of storing water behind dams is the reduction of adverse impacts caused by nature's extremes of flooding and drought. It is further stated that climate change causes changes in extreme precipitation and drought events resulting in increased hydrological risks. It is critical to consider changes in climate during planning and management of dams. This may result in a need to increase the height of dams, expand spillway capacity, modify reservoir operating procedures, and/or construct new dams [24]. The ICOLD's Technical Committee on Global Climate Change and Dams, Reservoirs and Associated Water Resources has been assessing the role of dams and reservoirs in adapting to the effects of global climate change, determining the threats, and potential opportunities, posed by global climate change to existing dams and reservoirs [24]. The dams-and-reservoirs based hydropower can play a crucial role in climate change mitigation. With ever increasing understanding of the causes and impacts of climate change, local, national and international agencies are recognizing the importance of climate change planning. As the frequency and severity of extreme events continue to intensify, infrastructure will continue to be tested and at times will fail. The U.S. Bureau of Reclamation has already initiated a series of pilot studies focused on exploring potential impacts of climate change on hydrologic hazards at specific dam locations. It reinforces the need to assess and address hazards created by climate change as part of the planning, design and operation phases [25, 30].

It is a matter of fact that climate change is causing more frequent extreme weather condition on a global scale. Due to this climate change, unprecedented extreme rainfalls and severe flood events can be expected, which are far more extreme than in past decades. Such situation can pose much greater uncertainty and risk on many existing dam projects. It is important that dam and river managing authorities take climate change related extreme floods into account when performing rehabilitation and/or reconstruction of existing dams. It has been known, for many decades, "climate change" is largely human-induced and is unpredictable over the time scale of a century. Key determinants of human-induced climate change are emissions of greenhouse gases, aerosols, and land-cover changes and it has been linked to higher precipitation and more heavy rainfall events in northern mid-latitudes. The risks of catastrophic flooding are not as rare as they once were and they will be more frequent in the future. Many managing agencies have already taken climate change into account in evaluating risk reduction measures concerning dam projects due to the greater degrees of uncertainty it creates. As in 2019, more than 91,460 dams are operated across the U.S. and, as of 2016, a total of 15,500 are classified as high-hazard dams [27].

Many dams are being reclassified due to growing concerns that many dam projects were developed based on 20th century assumptions and hydrological records that did not consider today's extreme weather conditions due to climate change. It is important for the managing agencies to raise awareness of the threat of extreme storms and promote emergency preparedness [21, 28].

Some facts of the climate change that are related to the Indian subcontinent are briefly described as follows. Every summer, monsoon rains come to southwestern India. In 2018, however, the deluge brought the worst flooding in a century. The resulting destruction killed nearly 500 people, inundated cities, and collapsed bridges. The rains also caused thousands of landslides in the mountains after torrents loosened soils from hill slopes. These slurries of water, soil, rock, and vegetation overwhelmed villages, downed power lines, and cut communities off from receiving aid. Then the monsoon returned with devastating force the following year, with the rains 400 percent above normal for a week in early August 2019. Back-to-back years brought flooding that would be expected only once every 100 years. [10]. The Indian summer monsoon brings torrential rains to Southeast Asia from June through September every year in a weather event with massive impacts on local economics, infrastructure, and culture. In summer winds from the southwest carry moisture from the Indian Ocean and deposit it over the Indian subcontinent. Around October, the winds reverse direction, and only certain regions receive the remaining moisture. The topography of the subcontinent further contributes to the intensity of the monsoon, with the Himalayas and the Western Ghats acting as barriers that trap severe weather above India. Despite its seasonality, the timing, duration, and intensity of the Indian Summer Monsoon vary widely [3]. Floods remain among the most widespread and impactful disasters, particularly in mountainous areas. Mountains represent areas of complex terrain, high relief, large precipitation variability, limited accessibility, and scarcity of hydro meteorological and pedagogical information. Hence floods in mountainous areas present unique challenges in flood forecasting, respond, and recovery. Flood generation processes are associated with localized, extreme events, such as cloudbursts, landslides, and glacier lake outbursts and are poorly understood. Since mountainous areas are particularly sensitive to climate change, their flood vulnerability will increase as changing precipitation, temperature, and snow and glacier extent affect the frequency and severity of many processes that trigger floods in mountains. [11].

The Indian subcontinent is particularly vulnerable to climate change because of its diversified socioeconomic and climate conditions. Climate change could affect monsoons, droughts, and glaciers in northern India. Changes in monsoon variability and glacier melt may lead to droughts over the Indian plains as well as extreme rains and abrupt floods in neighboring Himalayas [27]. The amount of warming due to climate change going into the deeper ocean is at record level [7], and its effect on glacier retreat is beyond imagination. Runoff from shrinking glaciers can both increase and decrease summertime stream flow. It also affects snow accumulation, evapotranspiration, and other hydrological factors that influence stream flow [12].

The impact of climate change has been experienced globally. The recent flooding in Venice, in 2019, was also linked to climate change. It was caused by a combination of high spring tides and a meteorological storm surge driven by strong sirocco winds. The resulting tide was highest in 50 years [14]. In the Far East, a group of Japanese researchers have noted that global warming could slow down the speed of typhoons passing over or near Japan, leading to great damage and wreaks greater havoc because the heavy rains and strong winds continue for a longer period [16]. The 10 years to the end of 2019 have been confirmed by several global agencies such as the World Meteorological Organization, the UK Met Office, NOAA, and NASA, as the warmest decade on record [6]. Climate change has been linked to Australian bushfires, record breaking temperatures in Europe, torrential rains in Indonesia, slower moving typhoons in Japan, more powerful hurricanes over southern U.S.A., and unusual cyclone activities over Indian Ocean. The temperature recorded on the land and sea surfaces also indicate the amount of warming going into the deeper ocean is at record levels. Warmer seas mean there is more energy available for tropical cyclones to form very quickly and intensify quickly. Scientists have indicated that, as Earth warms, there will be less frequent rain but more extreme precipitation with heavy downpours that dump abnormally large quantity of rain that can cause dangerous floods. It has also been predicted that precipitation and wind extremes will occur simultaneously as compound extremes more often causing more damages to properties and infrastructures. When the combination of two or more hazard events or climate variables leads to an extreme impact, the result would have a multiplier effect on the risk to, not just the dam projects but also to the society, the environment, and infrastructure [8].

It is important that potential extreme weather conditions related to climate change be taken into account in the dam safety risk assessment and the related hydraulic model studies. The primary objective of DRIP is to improve dam safety across India. As a part of its mission, a new set of guidelines for selecting and accommodating inflow design floods for dams have been prepared. With climate change becoming a significant factor in safety and risk assessments of dams, the inflow design floods of many dams in India are expected to be reviewed and modified. Most of the inflow design floods will likely be increased remarkably that will require serious review, reassessment, and hydraulic analysis of the existing hydraulic structures. Many hydraulic structures and conveyance system will likely be modified to achieve a fully operational and functional spillway which is critical to safe dam operations. It is foreseeable that hydraulic model study including physical and/or numerical model studies will play an important role in carrying out the mission on improving dam safety across India.

2. Modeling Methods for Analysis of Hydraulic Structures

In U.S., along with many other areas of America‘s infrastructure, there is a significant need for the evaluation and potential rehabilitation of dams across the country, the U.S. Federal Emergency Management Agency (FEMA) estimates that by 2020, 85 percent of large dams will have exceeded their design lifespan or soon thereafter. Furthermore the NRCS (Natural Resources Conservation Service, USDA) estimates that more than 600 watershed dams need to be upgraded to ensure the safety of those downstream. Typically the design life of dams is 50 years which combined with the fact that the peak construction period for these facilities was 1955-1965 makes this a critical issue. To manage and/or upgrade those aging dams in U.S. and other countries, advanced computational models are required to carry out various types of hydraulic analyses [30].

With significant advances made in modeling methods, the broadening hydraulic issues related to dam safety assessment are predominantly addressed by means of hydraulic modeling. Hydraulic modeling is widely used to investigate design and operation issues in hydraulic engineering. It entails, with a degree of sophistication that varies with objective of the investigation, the use of hydraulic models for replicating flow and fluid-transport processes in diverse natural flow systems and for evaluating the performance of hydraulic structures and hydraulic machines. The common subjects for modeling include: water movement and sediment transport in rivers and coastal zones; the hydraulic performance of water intakes, spillways, and outlets; flow around various objects; flow through, or in, various conduits or flow regulating devices; performance of turbines, pumps, and other hydro machines; performance of floating structures or ships; and effluent-mixing processes, etc. One of the major advantages of a hydraulic model is its potential capacity to replicate many features of a complicated flow situation. It can provide valuable insight for engineers to understand the flow characteristics with the aids of instrumentation. Development of a physical hydraulic model requires due consideration of similitude, appropriate or practicable model scales, model layout, and model use in conjunction with other approaches, such as numerical modeling and field studies. For flow processes occurring over a large area or over a long duration, modeling may be carried out effectively using a physical model in concert with a numerical model, utilizing the strength of both modeling methods [43, 44].

For many years physical hydraulic modeling has been a standard design tool for hydraulic engineering of dams and their associated flow control structures. Engineers 'ability to gather and analyses data from physical models continues to improve with the development of more sophisticated electronic instrumentation, faster computers and advanced software. These same advancements in computer technology have also enabled the development of advanced mathematical and numerical modeling techniques and many powerful one-dimensional, two-dimensional and three-dimensional simulation software systems are now commercially available as options to hydraulic engineers. Physical models are often complemented by numerical models to develop the design of a dam project. It is not uncommon for the design engineer to utilize a numerical model to help develop a proposed design for a hydraulic structure or to gain insight at the preliminary stage of a project. Numerical modeling can be cost effective and accurate when based upon quantifiable field or physical modeling data. When a numerical model is based upon sound prototype or physical modeling information, the numerical model can be used for years on end as a valuable design and operational tool. Yet numerical modeling is limited when it comes to multi-phase flows (air/water mixtures), highly turbulent flows with vortices and flows where scour and/or sedimentation are a concern. The current trend for modeling hydraulic structures is to utilize both numerical and physical models in parallel during a study (also known as composite modeling). Composite modeling is the effective utilization of the strength of both physical and numerical modeling methods, used either in series or in parallel with one another to solve difficult hydraulic problems.

Although physical modeling is the proven standard for modeling hydraulic structures and has successfully been used for decades, it comes with limitations and constraints. Similarly, numerical modeling, which is relatively new, also carries with it a number of limitations and constraints as well as benefits not found in physical modeling. However, when the two modeling techniques are used together during a hydraulic study, many of the limitations of one technique can be complemented by the other and vice versa. Therefore, when engineers understand the benefits and limitations of each modeling technique, they can develop a better plan that utilizes composite modeling approach where both modeling techniques are used together to increase the accuracy and efficiency of the modeling process. To reduce construction costs for the physical model and allow a largest possible size model to be constructed economically, and to reduce the Reynolds number scale effects in the region of interest, a numerical model study can be performed before the physical model is constructed and tested, the numerical modeling results can provide valuable information so that the approach geometry of the physical model can be properly aligned with the approach streamlines in the flow, and that the supplies water to the physical model is not constructed any larger than necessary.

In some situation it is desirable that numerical modeling is performed in parallel with the physical model. One of the most significant benefits of parallel composite modeling is "modeling the model", which means that the exact geometry of the initial or baseline physical model is numerically modeled at a 1:1 scale so that numerical modeling errors can be minimized. After every physical model study is completed, it is necessary to dismantle the model so that the space that was occupied by the model in the laboratory can be used for another project use. However, the numerical model does not take up any space and does not need to be discarded. The calibrated numerical model can be saved and maintained after the physical model study is complete and can be used indefinitely for future needs.

Additional information can be collected within the flow field (pressure and velocities) if needed after the physical model has been dismantled. Minor modifications to the geometric design can be made and tested in the calibrated numerical model with confidence [31, 43, 44, 47, and 48].

3. Application of Physical Models in Hydraulic Engineering

Summary

In this section, an overview of some of the basics of physical modeling as applied to hydraulic engineering and dam safety assessment is presented. Different ways to classify physical models including fixed- versus moveable-bed, two- versus three-dimensional, and site-specific versus generic models are described. The basics of hydraulic similitude and scale factors including derivation of Froude and Reynolds numbers and other dimensionless factors used in analyzing fluid flow problems are discussed. The selection of model scale factors and calculation of equivalent model and prototype scaled values for commonly used quantities are described. The advantages and disadvantages of physical models are discussed including consideration of laboratory and scale effects. Finally, the steps involved in the design and planning of physical models from start to finish are presented. [43-48].

Application of Physical Models

Traditionally, the design and operation of hydraulic structures has been accomplished through use of physical modeling. Physical models for dam engineering are concerned with the design and interaction of flow in rivers, lakes, and streams with dams, control structures, locks, and stream beds, etc. They are used to evaluate the multi-dimensional flow patterns associated with approach geometries, riverine bend ways, scour, internal hydraulics and cavitation, energy dissipation, etc. Nowadays, combinations of analytical, numerical, and laboratory modeling tools are typically used to ensure optimal designs.

Physical models have been used to study investigate concerns related to free-surface fluid flow problems often encountered during design and analysis of spillways and stilling basins, etc. Physical models are important engineering tools that are used to (a) improve understanding and gain insight into the hydraulic and riverine processes, (b) test and optimize proposed designs, and (c) provide data sets for improving numerical models. Physical systems that are reproduced in the model follow similarity laws derived from dimensional analysis so that the model accurately represents the prototype. In general, the ratio of like quantities should be the same in both the model and prototype. Use of physical models is a well-established engineering practice, especially in hydraulic engineering and the design of ships. Froude studied the resistance of ships in a towing tank in 1870, which involved inertial and gravitational forces and developed the Froude number. Reynolds developed the Reynolds number in his study of flow in parallel pipes to distinguish between laminar and turbulent flow conditions. The Froude and Reynolds numbers are two of the most important non-dimensional similarity parameters used in physical modeling. Figure D-1 is an example of a physical hydraulic model of the Dalles spillway that was used to optimize the proposed design modifications [43, 47]. Stability analyses of concrete gravity spillways are complicated by the hydrodynamic forces resulting from changes in flow speed and direction. The depth of flow, pressure distribution along the spillway profile, and tailwater conditions are not easily solved without rigorous analysis and physical model studies [115].

Figure D- 1 the site-specific, 3-D, fixed-bed, 1:80 scale model of the Dallas spillway.

There are several different ways to classify physical models. These include fixed- versus moveable-bed, two- versus three-dimensional, undistorted versus distorted, and site-specific versus generic classes. Fixed-bed models have solid boundaries (e.g., cement) that are not modified by the hydrodynamic processes being modeled. This is the most common type of model and the scaling effects are relatively well understood. Moveable-bed models, on the other hand, have soft boundaries that interact with the hydrodynamic processes. They are used to study transport issues such as river/channel profile evolution, erosion and scour processes around structures, and changes in bathymetry due to water flow. The scaling for moveable-bed models is not as well understood as fixed models.

Two-dimensional (2-D) models examine hydrodynamic processes in the dimensions of length and depth only. A typical 2-D laboratory facility is a flume or wave tank which is much longer than it is wide. Three-dimensional (3-D) model simulations include the effects of the third dimension of width on hydrodynamic phenomena. The 3-D models are more costly to build and test than 2-D models.

A model is undistorted if it has the same geometric scale in both horizontal and vertical directions. This is the usual practice for most models, especially those that involve waves as one wants to accurately model the water particle motions in both horizontal and vertical directions. A distorted model has different scales for horizontal and vertical dimensions. Open-channel flow models are typical of these types of models. The horizontal scale is much smaller than the vertical scale since these models cover a large area. As an example, a distorted harbor model may use a horizontal scale of 1:400 (model: prototype), and a vertical scale of 1:100.

Site-specific models are designed for a particular project site with corresponding forcing conditions. They are used to verify and validate design(s) for that specific site only. Generic models are usually simplified and idealized models intended to provide guidance for a variety of hydrodynamic processes. They can be used to investigate a particular response under controlled conditions, calibrate numerical models, or develop empirical relationships.

- Hydraulic Similitude: the purpose of hydraulic similitude is to ensure that the model reproduces the behavior of the prototype as much as possible. This similar behavior includes velocity, acceleration, mass transport, and resultant fluid forces on objects and boundaries. Correspondence between prototype (p) and model (m) is denoted by the scale factor or model scale (N_x), which is the ratio of the prototype parameter (X_p) to the model parameter (X_m) defined as:

$$N_X = \frac{X_p}{X_m} \qquad \text{Equation D-1}$$

Note that one can sometimes see the inverse of this factor used to represent the scale factor (i.e., model divided by prototype value). This makes the scale factor look like a fraction less than one, as in 1 model-100 prototype. The form given in Equation D-1, where the prototype value is divided by the model value, is preferred. This X parameter can represent any derived variable as determined from its dimensions. For instance, fluid velocity (V) has dimensions of length (L) divided by time (t), so the velocity scale (NV) is given by:

$$N_V = \frac{V_P}{V_m} = \frac{(\frac{L}{t})p}{(\frac{L}{t})m} = \left(\frac{L_p}{L_m}\right)\left(\frac{t_m}{t_p}\right) = \frac{N_L}{N_t} \qquad \text{Equation D-2}$$

The three basic laws of similitude are geometric (similarity of form), kinematic (similarity of motion), and dynamic (similarity of forces). In general, the ratio of quantities in the model needs to be the same as in the prototype. For geometric similarity, the ratio of model and prototype lengths must be equal. For kinematic similarity, velocity and acceleration must have the same ratios between model and prototype. For dynamic similarity, the four external forces of gravity (Fg), viscosity (Fv), surface tension (Fs), and elasticity (Fe) must have the same ratios. This requirement arises from Newton‘s Second Law which states that the inertial force (FI=ma) equals the sum of these external forces.

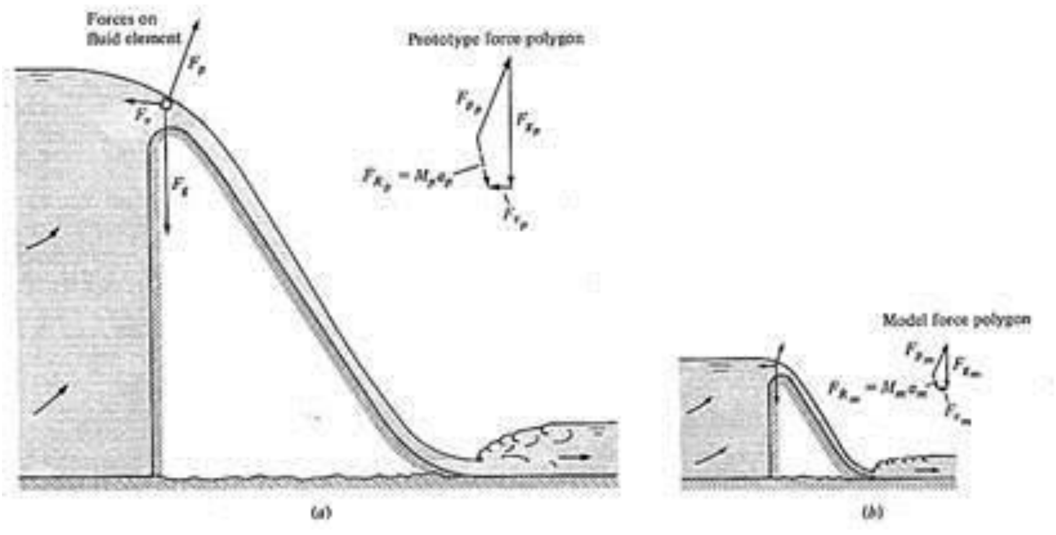


Figure D- 2 an illustration of the prototype-model relations.

As an example, one may consider the case of flow over a spillway as shown in Figure D-2. The corresponding masses of fluid in the model and prototype which are acted upon by corresponding forces. These forces are the force of gravity Fg, the pressure force Fpressure (FP in the figure), and viscous resistance force Fv. These forces are shown vectorially in Figure D-2 to yield a resultant force FR which will in turn produce an acceleration of the volume of fluid in accordance with Newton‘s second law. Hence, because the force polygons in the prototype and model are similar, the magnitudes of the forces in the prototype and model will be in the same ratio as the magnitude of the ma (Ma in the figure) vectors [33].

In general, almost any modeling situation can be simplified as the interplay between two major forces as the other forces play a minor role. In flow modeling, inertial forces are always present. The ratio of the inertial force to the other forces has led to the development of the similitude criterion that relates the importance of these secondary forces to the inertial force. No fluid can satisfy all the dynamic similarity requirements. Since the same fluid (i.e., water) is usually used for both model and prototype, it is impossible to achieve exact dynamic similarity for water waves.

Surface tension and compressibility are generally neglected as they are relatively insignificant. Viscosity can be neglected in most free-surface models if the model is not too small. Thus, the Froude Number is the major scaling criterion in physical hydraulic models with free-surface flows.

- Froude Model Law: the Froude number (F_r) is defined as the square root of the ratio of the inertial force (force due to convective acceleration of a fluid particle) to the gravitational force (weight) as given by:

$$\boldsymbol{F_r} = \sqrt{\frac{\boldsymbol{F_I}}{\boldsymbol{F_g}}} = \sqrt{\frac{\boldsymbol{\rho L^2 V^2}}{\boldsymbol{\rho L^3 g}}} = \frac{\boldsymbol{V}}{\sqrt{\boldsymbol{gL}}} \qquad \textbf{Equation D-3}$$

Froude similitude between model and prototype requires that the Froude numbers are equal:

$$\mathbf{F_r} = \left(\frac{\mathbf{V}}{\sqrt{\mathbf{gL}}}\right)_{\mathbf{m}} = \left(\frac{\mathbf{V}}{\sqrt{\mathbf{gL}}}\right)_{\mathbf{p}} \qquad \textbf{Equation D-4}$$

Rearranging and expressing in terms of scale ratios (NX) of individual variables gives:

$$N_V = \frac{V_p}{V_m} = \sqrt{\frac{g_p}{g_m}}\sqrt{\frac{L_p}{L_m}} = \sqrt{N_g}\sqrt{N_L} \qquad \text{Equation D-5}$$

- Reynolds Model Law: the Reynolds number (R_e) is important when viscous forces dominate, such as laminar boundary layer problems in open-channel or free-surface flows and forces on cylinders in low R_e flows. Bottom friction can be significant if the flow depth is relatively small. One way to minimize the influence of viscosity is to ensure that the model flow is in the turbulent range, which occurs for R_e above approximately 10^4. The Reynolds number (R_e) is defined as the ratio of inertial to viscous forces:

$$R_e = \frac{F_I}{F_V} = \frac{\rho L^2 V^2}{\mu V L} = \frac{\rho V L}{\mu} = \frac{VL}{v} \qquad \text{Equation D-6}$$

Where ϱ is the mass density, μ = dynamic viscosity, and ν (=ϱ/μ) is the kinematic viscosity. As done with the Froude number, similitude is achieved when Reynolds numbers are equal in model and prototype:

$$R_e = \left(\frac{\rho L V}{\mu}\right)_m = \left(\frac{\rho L V}{\mu}\right)_p \qquad \text{Equation D-7}$$

$$N_\mu = \frac{\mu_p}{\mu_m} = \left(\frac{\rho_p}{\rho_m}\right)\left(\frac{V_p}{V_m}\right)\left(\frac{L_p}{L_m}\right) = N_\rho N_V N_L \qquad \text{Equation D-8}$$

An interesting comparison is the possibility of satisfying both Froude and Reynolds criteria in an ideal physical model. The difficulty arises since gravity and water are typically constants in both model and prototype. Equating Fr and Re requires an impossible equivalence unless one uses a centrifuge or goes into space to alter gravity since there is no fluid that has a viscosity that can satisfy these constraints. In general, the viscous forces can be neglected if the Reynolds number is greater than 10^4 and in the same range in model and prototype, even though they are not exactly the same.

In addition to the Froude and Reynolds numbers, there are several other dimensionless numbers that can be derived for use in physical models. These include (a) the Weber Number (W_e) that is important in the study of surface tension, (b) the Cauchy Number (Ca) for the study of elasticity and compressibility in breaking waves, impact forces, and mooring lines, and (c) the Strouhal Number (S_t) for the study of acceleration effects in unsteady, oscillating flows, and vortices such flow through trashracks. Table D-1 lists Froude-derived scale factors for some common variables used in coastal and hydraulic engineering models. It includes units, type of similitude, and Froude-scaling relative to the model scale factor (λ). Note that λ is usually chosen based on the geometric similitude factor (N_L) for characteristic lengths. Also, it is assumed that gravity remains constant and fresh water is used in both model and prototype.

Variable	Units	Similitude	Froude Scale
Length	L	G	λ
Area	L^2	G	λ^2
Volume	L^3	G	λ^3
Angle	*deg*	G	1
Time	T	K	$\lambda^{0.5}$
Frequency	T^{-1}	K	$\lambda^{-0.5}$
Velocity	LT^{-1}	K	$\lambda^{0.5}$
Acceleration	LT^{-2}	K	λ
Discharge/Flow Rate	L^3T^{-1}	K	$\lambda^{2.5}$
Kinematic viscosity	L^2T^{-1}	K	$\lambda^{1.5}$
Mass	M	D	λ^3
Fluid Density	ML^{-3}	D	1
Dynamic viscosity	$ML^{-1}T^{-1}$	D	$\lambda^{1.5}$
Pressure/Stress	$ML^{-1}T^{-2}$	D	λ
Force/Shear/Weight	MLT^{-2}	D	λ^3
Moment/Energy/Work/Torque	ML^2T^{-2}	D	λ^4
Momentum/Impulse	MLT^{-1}	D	$\lambda^{3.5}$
Power	ML^2T^{-3}	D	$\lambda^{3.5}$
Notes: *M*=Mass, *L*=Length, *T*=Time, G=Geometric, K=Kinematic, and D=Dynamic similitude. With same specific weight of material, otherwise density N_ρ scaling is necessary.			

Table D- 1 Froude scale factors for commonly used variables

Some criteria for the selection of the model scale factor are listed as follows:

- Make the model as large as possible since the larger the model, the more accurate the results will be as the model behavior is most like the prototype.
- Compromise on size if necessary as larger models cost more and may require more time to complete an experiment.
- Consider the size and availability of test facilities.
- Maximize model dimensions and sizes of measured quantities to minimize viscosity and surface tension effects.

There is no definite criterion for acceptable ranges of scale factors. However, most hydraulic laboratories have had success with scales in the range of 10 to 150 depending on the parameters tested. The smaller values are usually in 2-D models to minimize the scaling effects in studies involving structural details such as breakwater armor units.

- Measurement Techniques: there are various measurement techniques available to carry out a physical model study. Figure D-3 shows the apparatus developed by Henri Emile Bazin for measuring pressure and velocity distribution through nappe in late 1800s.Today there are several classes of current meters available in water measurement. Such as anemometers and propeller velocity meters; electromagnetic velocity meters; Doppler velocity meters; and optical strobe velocity meters, etc.

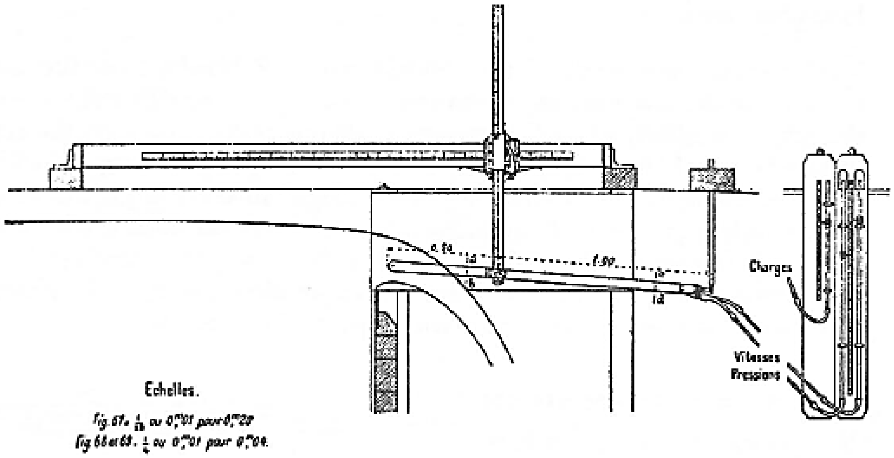


Figure D- 3 Bazin's apparatus for measuring pressure and velocity distribution through nappe for vertical weir.

Anemometer and propeller: current meters use anemometer cup wheels or propellers to sense velocity. The Price current meter and the smaller pygmy meter modification are the most common current meters in use. These meters are rated by dragging them through towing-tanks of still water at known speeds. The reliability and accuracy of measurement with these meters are easily assessed by checking mechanical parts for damage and using spin-time tests for excess change of bearing friction. This type current meter does not sense direction of velocity, which may cause problems in complicated flow where backflow might not be apparent. Some images of propeller current meters such as OTT C31 Current Meter; Dumas Neyrpic current meter; USGS pygmy current meter; and Price Type-AA current meter are shown in Figure D-4.

Figure D- 4 Images of propeller current meters (OTT C31, Dumas, SEBA F-1, & Price Type-AA).

- Electromagnetic current meters: produce voltage proportional to the velocity. One advantage of these current meters is direct analog reading of velocity; counting of revolutions is not necessary. These current meters can also measure cross flow and are directional. Their use near metallic objects is a limitation. Some images of electromagnetic current meters such as Marsh McBirney Flo-Mate 2000; Sanko Electro-Magnetic meter; Valeport - Model 803–ROV; and Valeport 801 electromagnetic current meter are shown in Figure D-5.

Figure D- 5Sample images of electromagnetic current meters: Marsh McBirney Flo-Mate 2000; Sanko electro-magnetic meter; Valeport - Model 803–ROV; and Valeport 801 electromagnetic current meters.

- Doppler type current meters: determine velocity by measuring the change of source light or sound frequency from the frequency of reflections from moving particles such as small sediment, suspended particles and air bubbles. Laser light is used with laser Doppler velocimeters (LDV), and sound is used with acoustic Doppler velocimeters (ADV). Acoustic Doppler Current Profilers (ADCP) has been used for deep flow investigations. These instruments measure average velocities of cells of selected size in a vertical series of a water column to measure vertical current profiles. ADCP measurements are becoming more frequent for deep flow in reservoirs, oceans, and large rivers. Most of the meters are multidimensional or can simultaneously measure more than a single directional component of velocity at a time. Some images of Doppler current meters such as SonTek Argonaut-ADV Doppler current meter; Nortek single-point Doppler current meter; Nortek compact Aquadopp ADCP profiler; and RDI Workhorse Sentinel ADCP are shown in Figure D-6.

Figure D- 6 Sample images of Doppler current meters: SonTek Argonaut-ADV Doppler current meter; Nortek single-point Doppler current meter; Nortek compact Aquadopp ADCP profiler; and RDI Workhorse Sentinel ADCP.

- Optical strobe velocity meters: use optical methods to determine surface velocities of streams. This meter uses the strobe effect. Mirrors are mounted around a polygon drum that can be rotated at precisely controlled speeds. Light coming from the water surface is reflected by the mirrors into a lens system and an eyepiece. The rate of rotation of the mirror drum is varied while viewing the reflected images in the eyepiece. At the proper rotational speed, images become steady and appear as if the surface of the water is still. By reading the rate of rotation of the drum and knowing the distance from the mirrors to the water surface, the velocity of the surface can be determined. The discharge rate of the stream may be estimated by applying the proper coefficient to this surface velocity and multiplying by the cross-sectional area of the flow section. The meter has several advantages. No parts are immersed in the flowing stream. Moreover, it can be used for high-velocity flows and for flows carrying debris and heavy sediment. The meter can measure large flood flows from bridges. However, the meter measures only the water surface velocity and is very dependent upon the selection of the proper coefficient.

Newer measurement techniques include surface Particle Tracking Velocimetry (PTV) for velocity measurements of the large-scale water bodies and Planar Concentration Analysis (PCA) / Laser Induced Fluorescence (LIF) for concentration measurements. For small scale test-facilities, Particle Image Velocimetry (PIV) and PTV measurements can also be performed. Any location in the test basin can be reached by a fully automatic 3-D traversing system on which additional point measurement systems can be mounted. Velocity measurement with multi-component devices include laser and acoustic doppler-velocimetry (LDA, ADV), laser tomography, particle tracking and image velocimetry (PTV, PIV), hot wire and hot film anemometry. The imaging-based equipment includes for example, Laser systems with various single PIV cameras. Measurement equipments may also include particle size analyzer, pressure and force transducers for hydraulic and soil mechanic parameters, and automatic experimental control and data acquisition with software such as Lab View etc.

In Figure D-7, images of water level sensors and loggers such as Global Water WL700 non-contact ultrasonic level sensor, Global Water WL-16 water level logger, and a Hoyt Level1000 water level & temperature recorder are provided. Images of water quality monitoring systems such as Hydrolab Quanta water quality monitoring system, Sequoia LISST-100 sediment size distribution sensor, and D&A OBS-3 turbidity& suspended solid sensor, and WQ710 turbidity sensor are shown in Figure D-8.

Figure D- 7 Images of water level sensors: Global Water WL700 non-contact ultrasonic level sensor; Global Water WL-16 water level logger; and Hoyt Level1000 water level & temperature recorder.

Figure D- 8 Images of Hydrolab Quanta water quality monitoring system, Sequoia LISST-100 sediment size distribution sensor, and D&A OBS-3 turbidity & suspended solid sensor; WQ710 turbidity sensor.

Advantages and Disadvantages of Physical Models.

Some advantages and applications of physical models are listed as:

1. Physical phenomena are often highly nonlinear and not well understood. Physical models usually replicate the physics, both linear and nonlinear, of flow processes well, without simplifying assumptions.
2. Physical models are used to determine empirical coefficients for analytical and numerical models that are not known or only poorly understood.
3. Physical models assist in evaluating the effect of assumptions on numerical model predictions.
4. Field experiments are expensive and forcing conditions (i.e., waves, winds, water levels, currents) are difficult to control and systematically vary.
5. Physical models provide repeatability and a controlled environment for calibrating analytical and numerical models. The smaller size permits easier and less expensive data collection of multiple variables such as water levels, pressures, forces, velocities, displacements, etc.
6. Physical models provide hands on look at the processes, allowing one to examine different options relatively conveniently and gain qualitative impression of the physics governing a process. It is easier for people to relate to the visual feedback and ability to interact with a physical model.
7. Prototype construction may be risky or uneconomical without a model to verify assumptions and performance. A physical model study is generally needed for verifying the prototype design and validating numerical model results.
8. Physical models can be used in conjunction with numerical models as a hybrid/composite model to take advantage of their individual strength and benefits. In a hybrid model, one can have numerical-physical-numerical or physical-numerical model connectivity. For instance, the numerical model is used to provide input to the physical model which then provides its output as input to the same or another numerical model.

Four main disadvantages of physical models can be listed as follows:

1. Scale effects are due to the fact that it is impossible to correctly scale a free-surface hydrodynamic model using water that satisfies all of the laws of similitude. To minimize scale effects, one should construct as large a model as possible that fits within time, cost, and available facility space constraints.
2. Laboratory effects consisting of model boundaries, instrumentation support, mechanical wave and flow generation losses, etc., are a concern in physical models. In the model, walls are necessary to contain the water, but they induce laboratory effects due to reflection and flow restriction. In the real world, there are no artificial boundaries to produce these effects. In the model, instruments are typically mounted in some type of rigid support that may have a larger impact on the flow field than a similar instrumentation support in the prototype. However, measures can be incorporated in the model to minimize and mitigate these effects. The generation of waves and currents by mechanical means in a laboratory is not exact between model and prototype. In the real world, waves may have multidirectional characteristics with frequency and directional spreading that are not always possible to simulate as accurately in the laboratory.
3. Forcing functions or boundaries in the prototype may not be simulated in the model due to cost and/or practicality. For instance, simulating wind blowing across the surface is

rarely included in coastal models. Instead, the wind effect is represented by the equivalent wave conditions at the wave-maker or wind effects on the model (i.e., ship heeling due to wind).

4. Construction and application of a physical model may not be cost-effective. For relatively straightforward applications, numerical models can be more cost efficient. One must realize that numerical models also have limitations and simplifying assumptions that may significantly affect the results. Tradeoffs between physical and numerical model applications should be considered.

Design and Planning of Model Experiments.

Proper planning is required to ensure a successful physical model study. Engineers need to be aware of the limitations and simplifying assumptions of the physical model. The model scale is an important consideration during all stages of the design and planning. It is recommended that a test plan be written prior to beginning the model to ensure that all details have been considered in the design and scope. The first step and most important consideration should be to determine a scale factor or range of scale factors that will satisfy the similitude requirements to solve the problem and/or answer concerns. The scale factor should be fine-tuned based on additional consideration of available test facilities, model construction costs, instrumentation, budget, and scope. The model design and layout must consider orientation so that all the desired forcing functions interact with the model. The time to construct the model must be considered in the project timeline. The instrumentation must be selected and positioned to ensure accurate measurement of desired responses without undue interference from mounting apparatus that could affect the measurements. Prior to production tests, the instrumentation and forcing generation equipment needs to be calibrated to ensure accurate simulation and measurement of the forcing mechanisms. This can be an iterative process as previous test results can be used to improve subsequent measurements. Finally, it is important to document and archive the study data and results, including all of these steps.

4. Examples of Designing Spillways using Physical Models

Physical Modeling of the Gross Dam Stepped-Spillway

The reservoir impounded by Gross Dam provides water to more than 1.4 million residents along Colorado‘s Front Range. As a part of the planned upgrade to Gross Dam by Denver Water (a public utility), the dam is to be raised 131 feet over its current height of 340 feet, increasing the capacity of Gross Reservoir by about 25 billion gallons. When complete, Gross Dam‘s will be the tallest stepped spillway in the United States (Figure D-9). The new spillway was design to have the ability to handle a major influx of water from a severe storm.

Figure D- 9 The raising of the Gross Dam and the proposed stepped (left), the 1:24 scale physical model of the stepped-spillway being tested at CSU (center & right).

The planned height of the raised dam necessitated the incorporation of a stepped-spillway. The spillway is the only portion of the dam over which water passes. Colorado State University's Hydraulics Laboratory was commissioned, in September 2019, to test the capacity, flow rate and storm readiness of the proposed stepped spillway by creating a 20-foot-high 1:24 scale model of the heightened dam‘s spillway (Figures D-9). The stepped-spillway dissipates energy from the water as it flows over the dam with the steps slowing the water, trap air bubbles, and allow water to safely descend. The spillway is designed very conservatively and must perform safely when exposed to extreme storms. Although ―computer models have come a long way, but they‘re not even close to being able to resolve what‘s happening in terms of interaction of forces, turbulence and air entrainment which are very hard to model accurately. It was therefore determined that a project of this type and magnitude requires a physical hydraulic model [36].

Oroville Dam Spillway Reconstruction Model study

The Oroville dam project which was built in 1968 consists of a 235-meter tall earth- and rock-fill embankment dam, an open chute service-spillway with control-gates at the crest and an emergency-spillway over natural hillside. The release of water through the service-spillway caused severe damage with a gaping hole formed on the concrete surface and failed quickly. Water was then sent over the emergency-spillway concrete crest and over a bare hillside, which caused near-catastrophic erosion and forced the evacuation of more than 200,000 people from downstream communities. It almost triggered a breach of the main dam. Although the weather condition in early 2017 was highly unusual, Professor Robert Bea (a professor emeritus of engineering at UC Berkeley) in his assessment of the causes of the spillway failure had concluded that the failure was caused by the dam managers by ignoring long-established guidelines and neglecting their duty to manage risks and detect flaws.

To quickly repair the tallest dam in America, it helps to shrink the problem to a more manageable size. Therefore, the water officials decided to use a physical scale model of the damaged Oroville to plan their repairs. The goal of this modeling effort was to help avoid a repeat of the disaster that unfolded in February 2017, when massive water releases from the reservoir caused the spillway to break apart, prompting evacuation of nearly 200,000 people downstream as a precaution.

Figure D- 10 The 1:50 scale model of the damaged Oroville Dam spillway.

A 1:50 scale model (Figure D-10) was built at the Utah Water Research Laboratory to help test repair plans before re-construction. The model was constructed out of wood, steel, concrete and acrylic plastic. The model used clear acrylic plastic for the gates and the spillway chute surface, because this material effectively simulates the friction between water and concrete at full scale. Water for the model was drawn from the near-by Logan River into a giant cistern beneath the warehouse that holds the model. Pumps circulate water through the model and return it to the cistern, allowing it to be used repeatedly. At 120ft (36m) long and 30ft (9m) wide, it is roughly the size of a tennis court. More than 50 test runs were made on the model to simulate the hydraulic conditions and to estimate the hydraulic forces acting on the spillway structure. The model helped fine-tune the rebuilt structure‘s design before a single bucket of concrete is poured.

The investigators also modeled the damaged areas of the spillway and the eroded hillside beneath it. They replicated those conditions using measurements taken by the water management agency using laser imaging and computer-aided design programming.

The purpose of modeling the damaged areas is to simulate what might happen if a freak storm requires massive water releases while repairs are under way, or if DWR (California Department of Water Resources) can‘t complete the repairs in time for next winter.

"No computer models can give all the answers, since" a "physical model is really an inexpensive insurance measure that can be taken to resolve some of the unknowns. It takes some of the guesswork out of the end product." Such scale models are commonly used when building a new dam to help verify calculations about water flow, depth and pressure. In fact, a model was built of Oroville Dam prior to its original construction in the 1960s. The new model simulates Oroville‘s entire main spillway, including the water-control gate structure at the top and a portion of the reservoir profile upstream. It does not include the adjacent emergency spillway.

The model was later converted to replicate the finished repairs, and tests were conducted to verify the final design. The project manager for the Oroville emergency spillway recovery effort said the physical model had helped verify computer and mathematical modeling already completed. It had shown, for instance, that the spillway‘s existing side walls are large enough to contain maximum water releases from the reservoir. The model was later used to test a new design feature that may be added to the spillway during repairs: aeration slots. These were cutouts in the concrete surface designed to prevent cavitation, a process that can erode concrete during high water flows. Aeration slots have been added at numerous other large dams as a safety measure [37-40].

Hydraulic model studies for Tala dam spillway, Bhutan

The Tala H.E. Project, Bhutan envisages construction of a 91 m high concrete gravity dam across river Wangchu near Honka and an underground 6-units powerhouse with installed capacity of 1020 MW. The spillway (Figure D-11) consists of several low-level sluices has been provided in the central portion of the dam. An overflow portion near the left bank has also been provided for passing floating debris. The sluices and the overflow portion are equipped with radial gates for passing floodwaters. Ski-jump bucket has been provided for energy dissipation for both the sluices and overflow spillway. The spillway was designed to pass standard project flood (SPF) with peak outflow of 8575 m^3/s at FRL El. 1363 m and PMF with peak outflow of 10,600 m^3/s at reservoir level El. 1365 m.

A 1:60 scale model (Figures D-11 & D-12) incorporating the spillway and river reach up to 900 m upstream and 1100 m downstream of dam was constructed. Hydraulic model studies were conducted for observing the flow conditions upstream and downstream of the spillway, discharging capacity, water and pressure profiles for various discharges and operating conditions of spillway and performance of the energy dissipater. The study demonstrated that the timely reference for conducting hydraulic model studies has helped in evolving the efficient hydraulic design of spillway, energy dissipater, and intake and plunge pool. The major modifications suggested during the model studies were introduction of curvature in dam axis, increasing number of spans from 4 to 5, introduction of divide walls, shifting of overflow spillway from right to left and location of the plunge pool. Also, during the course of study the top profile of the sluice spillway was modified to minimize cavitation impacts. The above study also highlights the importance of hydraulic model studies to be conducted during the design stage for accelerated development of hydropower projects. [53].

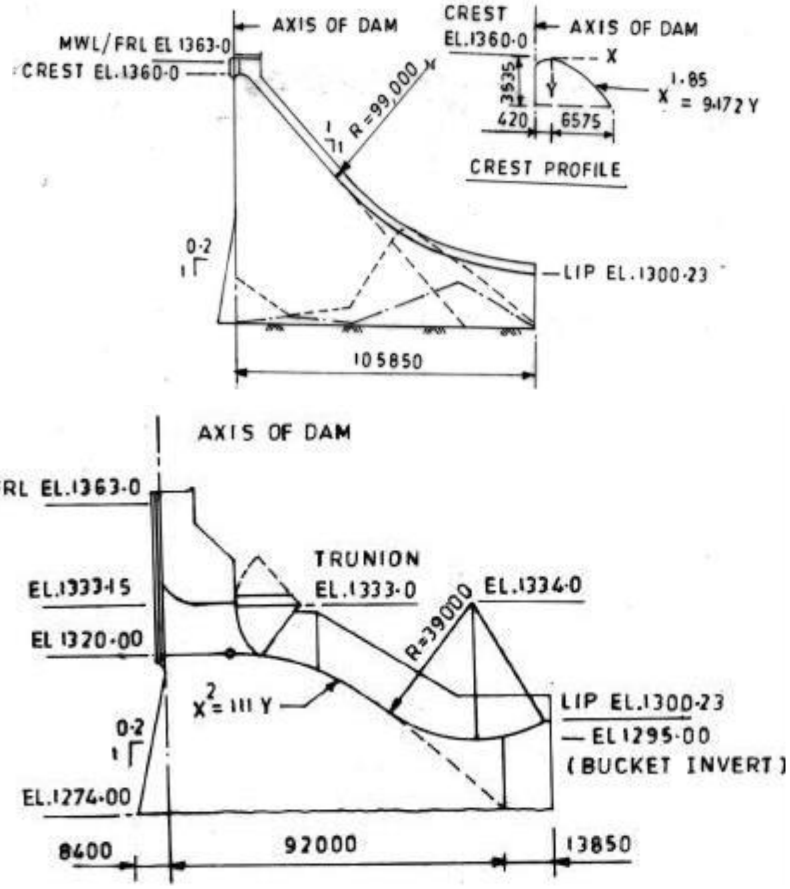


Figure D- 11 Sectional views of modified sluice and overflow spillway.

Figure D- 12 Model tests showing flow conditions downstream of spillway with (left) and without (right) divide walls.

Physical Model Study of Spillway and Energy Dissipation Arrangements of Malana Dam, Kullu, India

Malana-II Hydro Electric Project (HEP) is a run-of-the-river scheme on the Malana river in the Kullu district of Himachal Pradesh, India. It utilizes a gross head of about 626 m and generates 100 MW of power. A concrete dam with top level of 2545 m is constructed across the Malana river. Two under sluice type spillways of total discharging capacity of around 788.18 m^3/s at maximum water level (MWL) of 2545 m are provided to spill the flood water. The estimated probable maximum flood (PMF) discharge is 650 m^3/s (Ahmad 2018).

On tripping of both units of the project on 24th August 2013, water level in the reservoir which was initially at minimum drawdown level of 2528 m starts rising. The project authorities tried to open the radial gates of under sluice spillways, however, they could not be raised. As a result, the reservoir level rose and overtopping of the dam happened which continued for about 4 hour. No major damage to the dam was reported, however, to ensure safety of the dam in future, an overflow spillway in the Dam was planned so that water can safely pass under inoperative condition of one gate which can be attributed to emergency situation for mechanical and human failure. The proposed overflow spillway was consisted of seven bays - six bays of 6.25 m wide and one bay of 4.5 m wide by converting the existing non-overflow blocks into overflow blocks with spillway crest at El. 2543.0 m. Plan and section of the existing dam are shown in Figure D-13 while modified plan layout of the dam, incorporating overflow spillway is shown in Figure D-14.

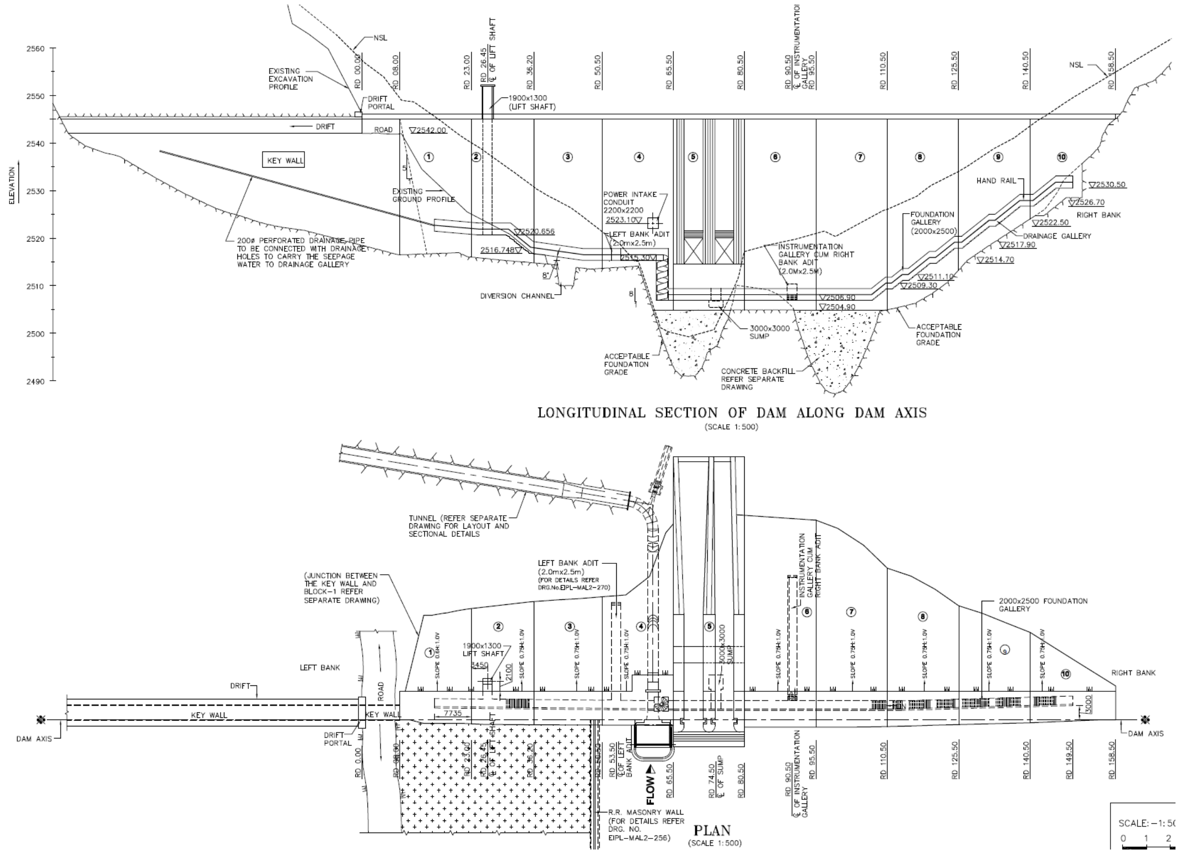


Figure D- 13 Plan and section of the existing dam and spillway

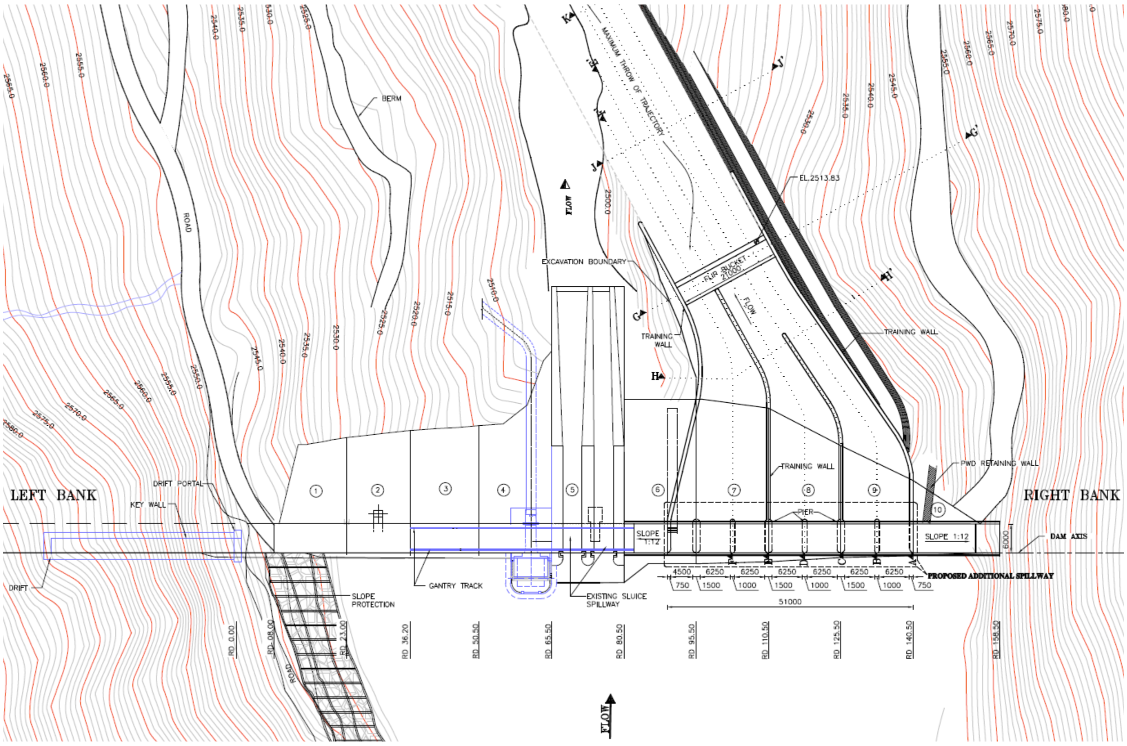


Figure D- 14 Plan layout of the modified dam and spillways

A comprehensive and geometrically similar model of the Malana-II hydro electric project dam and its spillways was built to a scale of 1:40 invoking the Froude number similarity for studying the discharging capacity of the spillways, cavitation over surface of the spillways, and energy dissipation downstream of the spillways. The model study was conducted under different operating conditions of the spillways i.e., (a) Both the under sluice spillways opened, (b) Only left under sluice spillway opened, (c) Only right under sluice spillway opened , (d) Only flow overflow spillway operational, (e) Left under sluice spillway and overflow spillway operational and , (f) Right under sluice spillway and overflow spillway operational. The discharging capacity of joint operation of both the under sluice spillways was much higher than the PMF discharge. Further, discharging capacity of either of under sluice spillway and proposed over flow spillway pass discharge higher than the PMF. The ogee profiles of the existing under sluice spillways and the proposed overflow were in order from the cavitation considerations.

Flow onwards to the toe of the overflow spillway was chaotic and full of strong cross-currents as shown in Figure D-15 (a) & (b). The flip bucket was also not performing well due to rise in depth of flow and mixed directional flow as a result of cross-currents upstream of the bucket. Thus there was a need to make changes in the stilling basin, training walls, side walls and flip bucket of the overflow.

For controlling the cross currents, lower portions of the proposed divide wall between block No.7-8 and Block No. 8-9 were removed. In addition, the super elevation in the sloping glacis of the right bay spillway block was also removed. Improvement in the flow pattern was observed, however, the cross currents were persisted. It was difficult to control the cross currents/waves due to formation of multiples cross currents in the presence of various concave and convex boundaries in the form of side walls and training walls. At outset, it was decided to replace the flip bucket arrangement with hydraulic jump type stilling basin by pooling the water with provision of a submerged broad crested weir of height 3.5 m along with steps downstream of the weir. Such arrangement controlled the cross-currents, however, still a strong cross current from the toe of dam block no.6 at the left end wall was impinging the pooled water. It was decided to shift the left side wall at the toe of the dam glacis by 3.0 m towards left side to minimize the formation of the cross-currents. L-section of the modified stilling basin of overflow spillway is shown in Figure D-16.

Figure D- 15 (a) Flow over spillway profile (b) Strong cross-currents in the stilling basin

The suggested changes in the over flow spillway were incorporated in the model. Figure D-17a depicts the modified stilling basin under no flow condition while Figure 17b shows for flow condition . Suggested changes comprise replacement of flip bucket by 3.5 m high broad crested weir with pool with steps cascade downstream of the weir. Each step is having 5 m width and 1.0 m drop and at the end of the first two steps, an end sill of height 0.5 m is provided. Left side wall was shifted by 3 m further left side near the dam toe. The width of the channel was kept 22 m in the place of earlier 21 m at the broad crested weir.

The model was run with exclusively over flow spillway for discharges of 101.19 m^3/s, 182.15 m^3/s and 263.10 m^3/s. The under sluice spillways were kept closed. Flow pattern in the stilling basin and over the submerged weir was observed during the run. Due to pooled water in the stilling basin with the provision of submerged weir, a relatively stable hydraulic jump was formed. The cross-currents were merging into the pooled water and losing their high kinetic energy. Flow in the stilling basin was highly turbulent with large size rollers which leaded to energy dissipation in the stilling basin. Flow over the submerged weir was almost uniformly distributed across the width of the channel and was falling onto the each step downstream of the weir. Overall, flow in the stilling basin and over the submerged weir was favorable from the hydraulic considerations.

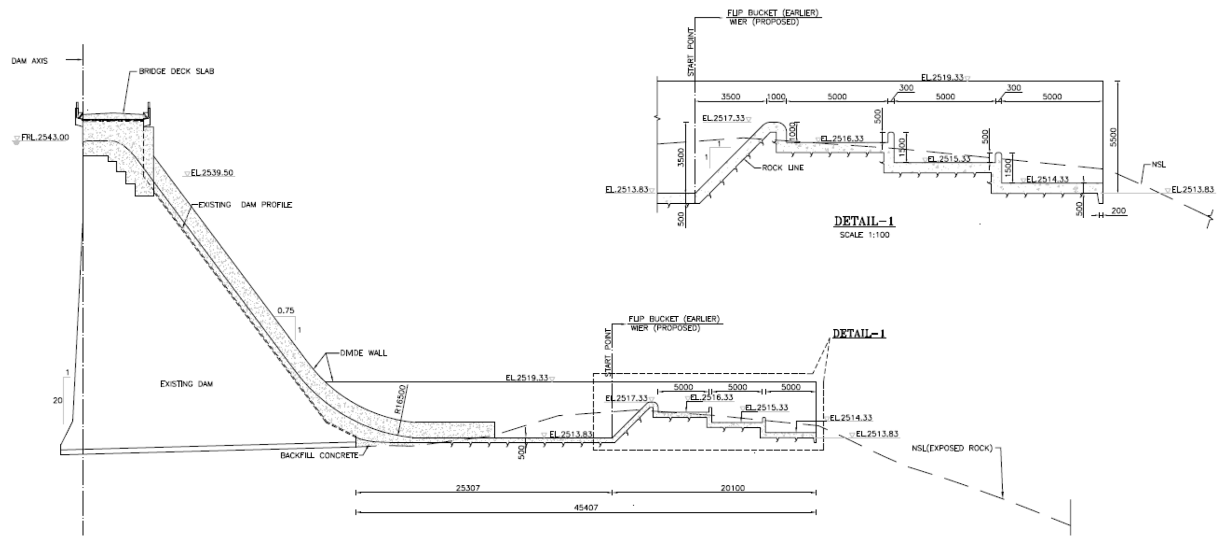


Figure D- 16 L-section of the modified stilling basin of the overflow spillway

Figure D- 17 Modified stilling basin under a) no flow condition; b) flow condition

- Stilling basin for flip bucket shall be designed in such a way that approaching supercritical flow should be free from the cross-currents to have sound flip bucket action. However, if cross-currents persist despite of provision of trailing walls, it is suggested to explore some other types of energy dissipators and be tested in the physical model.
- In the case of multiple intakes/under sluices provided a dam, operation of one affect the discharging capacity of the other if not placed sufficiently apart. This aspect is to be investigated in depth to quantify the interference effect.

Using Physical Hydraulic Model Studies to Quantify Rock Scour Extent

Bluestone Dam (Figure D-18) on the New River, West Virginia was subject to increased design flows. The dam spillway was modified to enable the operation of up to six penstocks. These penstocks provide an auxiliary discharge mechanism to increase discharge capacity during operational and flood conditions. During the construction period of the energy dissipating spillway, the operation of these penstocks exposed the bedrock formation downstream of the penstock stilling basin to scour. In order to assess these conditions, the USACE commissioned a study to investigate the potential for, and extent of, bedrock scour downstream of the penstock stilling basin. Rock scour potential and maximum depth were estimated using Annandale‘s Erodibility Index Method [144-145]. This method, in conjunction with hydraulic data collected from two physical scale models, provided a means of quantifying the erosion threshold of the downstream bedrock and the hydraulics of the highly turbulent tailwater. The stream power of the flowing water was quantified based on pressure fluctuation measurements taken at the bed of two physical scale models under design conditions. Maximum scour depth was determined by measuring the change in the applied stream power during scour-hole development, relative to the bedrock's ability to resist scour. It means that the pressure fluctuation measurements collected from the models were converted to stream power for estimation of the maximum scour depth. [142-145].

Figure D- 18 The Blue Dam project, the penstock outlets and stilling basin.

Bonneville Powerhouse ERDC Turbine Model Operating Range Investigation

The Bonneville second powerhouse turbine model investigations were performed to evaluate juvenile fish passage through turbines, with emphasis on identifying turbine structures and operations responsible for fish injury. A 1:25 scale physical hydraulic model of the turbine (Figure D-19) was used. Results were used to develop a target operating range for safer passage of juvenile fish. The study evaluated passage through the stay vane and wicket gate cascade (distributor), the turbine runner, and the draft tube. The study also evaluated retention time within the immediate tailrace region as influenced by draft tube exit conditions. Results from this model investigation indicate operations with steeper blade angles, resulting in higher unit discharge, create better passage conditions for fish. Fewer severe contacts with the turbine runner and less severe change in direction occur at steeper blade angles. The draft tube performs better at steeper blade angles, and the quality of flow exiting the draft tube is better at steeper blade angles.

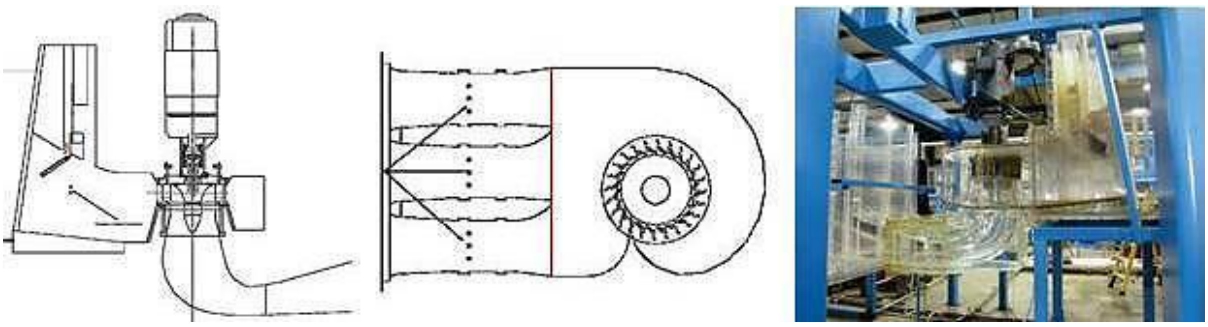


Figure D- 19 Bonneville second powerhouse turbine model with egress bead release locations and view of model structure and motor.

Data acquisition was accomplished using Laser Doppler velocity (LDV) meter. A four-beam, two-color LDV was used to measure draft tube velocities. This system consisted of a 4-watt Argon laser, optics to split and color separate the laser beam according to precise frequencies of light, fiber optics to carry the light to the model, a fiber probe with a 23.6 in. focal length, and signal processors for analyzing the signal from the fiber probe. A computer-controlled traversing system precisely controlled the position of the fiber probe for each velocity measurement. This system measured two components of the flow field within a plane perpendicular to the light beam. The LDV system is a non-invasive tool for defining flow distributions, velocity magnitudes and direction, turbulence, and investigations of hydraulic shear. The system was also used to determine the effects of the draft tube exit flow characteristics on egress conditions. The velocity data were used to document the flow split between the two draft tube barrels as well as provide a barrel turbulence intensity value for each barrel and the uniformity of flow within and exiting each draft tube barrel.

Neutrally buoyant beads were released within the intake at numerous locations, and their trek was documented through the stay vane and wicket gate cascade, runner, draft tube elbow, and tailrace region using high speed cameras. Frames rates between 120 and 1,000 frames/sec were utilized depending on the region being studied. Beads were evaluated for surface contact and sudden changes in direction (shear) as well as resident time durations in the tailrace. In addition, laser Doppler velocity measurements were obtained within the draft tube near its exit. The captured images of the bead distributions exiting runner 26.5 deg and 16.5 deg blade angles with beads released in all bays at screen tip are shown in Figures D-20 (a) and (b) respectively. A similar turbine model study (Figure D-21) was also conducted for the Bonneville first powerhouse [41-42].

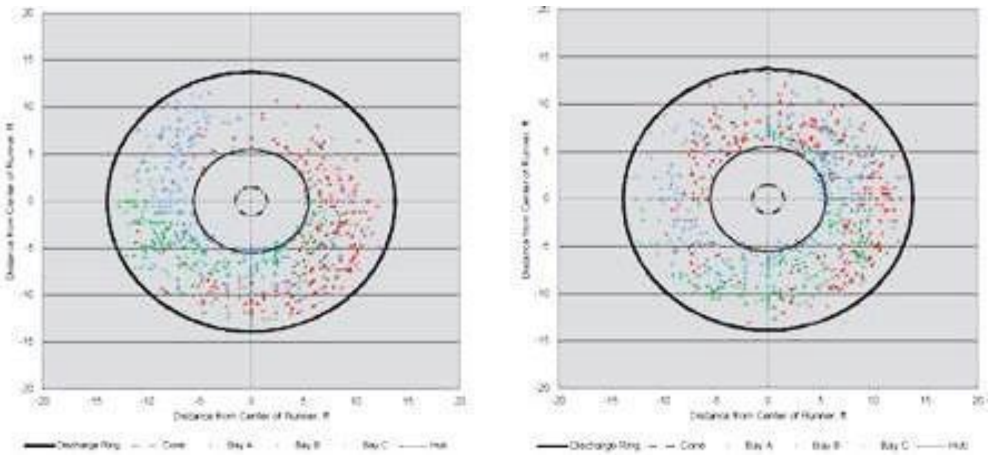


Figure D- 20 The images of the bead distributions exiting runner 26.5 deg (a) and 16.5 deg (b) blade angles.

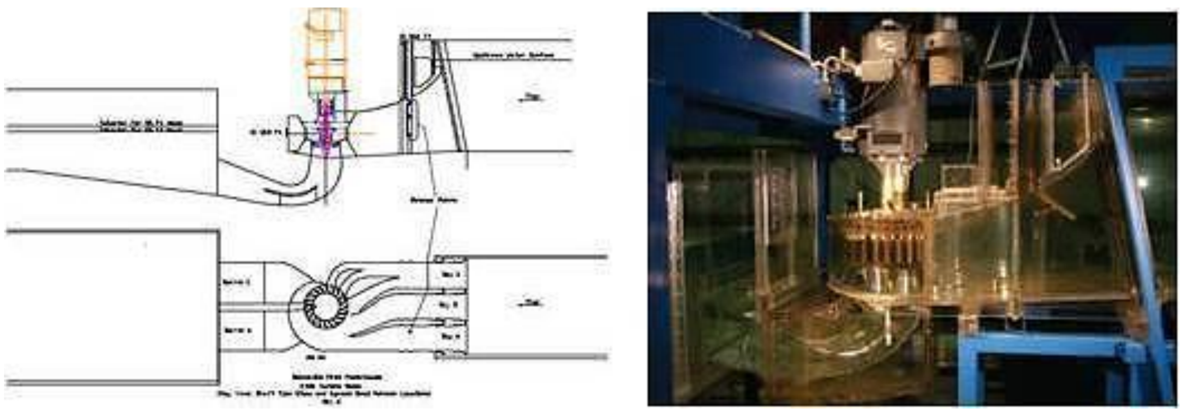


Figure D- 21 Bonneville first powerhouse 1:25 scale turbine model.

A sample observed horizontal velocity component distribution for flow near the draft tube exit under 55 ft head with 27.5 deg blade angle for the first powerhouse is shown in Figure D-22.

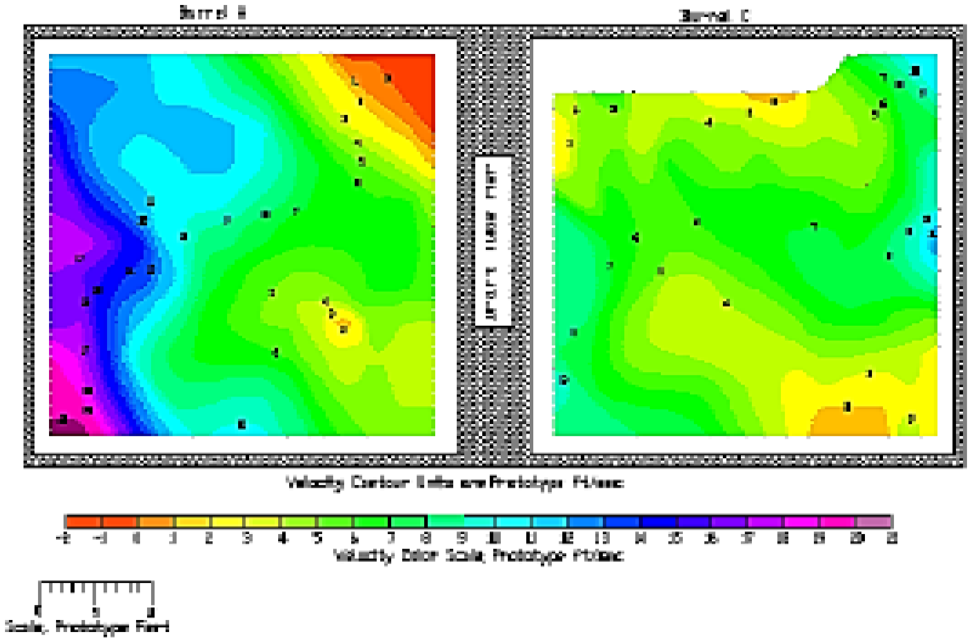


Figure D- 22 A sample observed horizontal velocity component distribution for flow near the draft tube exit of the first powerhouse.

Other examples of Designing Spillways using Physical Models

Spillways and outlets for river diversion and Normal Operation of Lower Caroni Hydroelectric Development (Appendix B) that comprises 4 large hydro power stations for a total of 17600 MW, Guri(10700 MW), Macagua (2970 MW), Caruachi (2260 MW) and Tocoma (2160 MW, yet under construction) and spillways with discharges of 30,000 m^3/s have been designed and optimized in the CVG EDELCA Hydraulics Laboratory, in operation since 1975 by using large comprenhensive 3D physical models built to Scales 1:50 to 1:100, and Sectional models built to scales (1:20 to 1:10).

5. Some Images of Physical Modeling Facilities at Several Hydraulic Laboratories

Central Water and Power Research Station, Pune, Maharashtra, India

Figure D- 23 Some model studies carried out at Central Water and Power Research Station, India.

Civil Engineering Department, Indian Institute of Technology (IIT), Roorkee, India

Figure D- 24 Hydraulics laboratory at Civil Department, Indian Institute of Technology (IIT), Roorkee, India

Utah Water Research Laboratory, Logan, Utah, USA

Figure D- 25 Some model studies carried out at Utah Water Research Laboratory, Utah State University.

CVG Edelca (Corpoelec) HydraulicsLaboratory – Venezuela

Figure D- 26 Some model studies carried out at EDELCA Hydraulics Laboratory.

Hydraulics Laboratory at Colorado State University, USA

Figure D- 27 Hydraulics laboratory facilities at Colorado State University.

Karlsruhe Hydraulics Laboratory-Institute for Hydromechanics, Germany

Figure D- 28 Laboratory facility at Karlsruhe Hydraulics Laboratory-Institute for Hydromechanics.

Institute of Hydraulic Engineering and Water Resources Management RWTH Aachen University, Germany

Figure D- 29 Hydraulics laboratory at Institute of Hydraulic Engineering and Water Resources Management, Aachen University.

6. Physical and Numerical Hydraulic Model Studies

Hydraulic model studies are commonly carried out for any significant hydraulic structure rehabilitation or modification projects. In recent years, both traditional physical scale model study and numerical model study are used to complement each other in many dam safety improvement projects. In many cases, field observations and data are used to develop these models. It is also common for investigators to use both physical and numerical models to design or modify hydraulic structures. Both types of modeling can be carried out in parallel or in sequence and it is called a hybrid or composite modeling approach. Physical models are also used to verify numerical models for simulations including prototype-scale analysis. In some cases, only physical model studies are carried out. However, many numerical models have been used for conducting preliminary investigations.

Depending on the project requirements and the objective of the investigation, a physical model can be 1-D, 2-D, or 3-D. It also can be distorted (meaning the horizontal scale is different from that of vertical) or undistorted.

Practical considerations dictated by model size and modeling convenience may require river models to be distorted. Generally, the upper limits for the geometric or length scales of a physical model are limited by the physical properties of water as the model fluid; in particular, viscosity and surface tension. Available floor space and water discharge capacity usually set the lower limits of model scale. To satisfy constraints on both the upper and lower limits of model size, it may be necessary to design a model using two geometric scales; that is, use a smaller scale for vertical lengths than for horizontal lengths. For bigger models encompassing large areas of shallow flow may require larger distortion. The main advantages of vertical distortion include reduced expenses incurred in construction and operating a model, and increased accuracy of flow-velocity and depth measurements in the model. When using models with vertical distortion cautionary factors such as exaggeration of secondary currents and distortion of eddies should be considered.

The Mississippi River Basin Model (by the USACE Waterways Experiment Station), located near Clinton, Mississippi, was a large scale distorted hydraulic model of the entire Mississippi River basin, covering an area of 200 acres (Figure D-30). The model was in operation from 1949 until 1973 and the scale of the model was 1:100 vertical and 1:2000 horizontal. The model study was conducted to control floods. It began with the Great Mississippi Flood of 1927, the worst natural disaster the country had known until that time. In response to the disaster, Congress passed the Flood Control Act of 1928, which authorized the U.S. Army Corps of Engineers to design and construct projects to control floods on the Mississippi River and its tributaries (as well as on the Sacramento River in California). Unfortunately, a decade later, the 1937 flood along the Ohio River inflicted new damage to the Mississippi River system. With two such significant floods on record, the USACE decided that a more comprehensive approach to flood control should be evaluated. A to-scale physical model of the Mississippi River system was needed as a critical tool for demonstrating potential flood-control measures to officials, laypeople, and engineers. It was one of the most successful experiments in physical hydraulic engineering ever constructed in the U.S. and was selected as an ASCE Historic Civil Engineering Landmark in 2018 [35]. Another large distorted physical model was the Chesapeake Bay model built by USACE. The model was built on a 60-acre site and the area modeled includes the bay and all its tributaries, up to the head of tide. At a scale of 1:1,000, 4,400 square miles of area was represented by eight acres of cement. The vertical scale was ten times larger than the horizontal at 1:100. The land area between the bay and river channels, the "overbank," were duplicated up to an elevation of 20 feet above mean sea level, just a few inches on the model.

Figure D- 30 The Mississippi River Basin Physical Model.

The Mississippi River Basin Physical Model (by the USACE Waterways Experiment Station), located near Clinton, Mississippi, is a large scale distorted hydraulic model of the entire Mississippi River basin, covering an area of 200 acres (Figure D-30). The model was in operation from 1949 until 1973 and the scale of the model was 1:100 vertical and 1:2000 horizontal. The model study was conducted to control floods. It began with the Great Mississippi Flood of 1927, the worst natural disaster the country had known until that time. In response to the disaster, U.S. Congress passed the Flood Control Act of 1928, which authorized the U.S. Army Corps of Engineers to design and construct projects to control floods on the Mississippi River and its tributaries (as well as on the Sacramento River in California).

Unfortunately, a decade later, the 1937 flood along the Ohio River inflicted new damage to the Mississippi River system. With two such significant floods on record, the USACE decided that a more comprehensive approach to flood control should be evaluated. A scale physical model of the Mississippi River system was needed as a critical tool for demonstrating potential flood-control measures to officials, people and engineers. It was one of the most successful experiments in physical hydraulic engineering ever constructed in the U.S. and was selected as an ASCE Historic Civil Engineering Landmark in 2018 [35]. Another large distorted physical model was the Chesapeake Bay model built by USACE. The model was built on a 60-acre site and the area modeled includes the bay and all its tributaries, up to the head of tide. At a horizontal scale of 1:1,000, 4,400 square miles of area was represented by eight acres of cement. The vertical scale was ten times larger than the horizontal at 1:100. The land area between the bay and river channels, the "overbank," were duplicated up to an elevation of 20 feet above mean sea level, just a few inches on the model.

Numerical models can also be models simulating one-, two-, or three-dimensional flows. One-dimensional numerical models are generally used to analyze hydraulics of large river systems and water conduit systems. A 2-D depth-averaged (integrated) simulation approach can be adopted when the flow is essentially two-dimensional in the horizontal plane. A 2-D width-averaged simulation approach can be considered when the lateral variations of the flow are insignificant. For simulation of reservoir dynamics, a 1-D (vertical) simulation model based on the physics-based parameterization approach has been successfully applied in many reservoir projects. There is a wide variety of numerical analysis techniques available to develop a numerical model such as finite-difference, finite elements, volume-of-fluid, and method of characteristics, etc.

Physical and numerical modeling should not be viewed as conflicting methods of investigation; rather, they have complementary strengths and weaknesses. Often a hydraulic engineering problem will require a combination of these methods, i.e. hybrid modeling, to achieve a cost-effective solution [32, 47, 48, 52, 86]. The following examples provide useful insights on how hydraulic model studies, utilizing different modeling techniques/approaches, have been conducted in association with dam safety assessments and hydraulic structure modification designs.

7. Examples of Hybrid/Composite Hydraulic Model Studies for Spillway Projects

Model Studies for a SHEM Spillway Rehabilitation Project

The dam is operated by the French electricity production company "Société Hydro-Electrique du Midi (SHEM)." The name of the dam and its location are not known. An update of the site hydrology suggested that the spillway discharge capacity should be increased to maintain the safety of the dam.

The hydraulic model study for developing the modification design of the spillway, as shown in Figure D-31 (b), was performed by the Hydraulic Engineering Laboratory of the University of Liege. A hybrid/composite hydraulic modeling approach was applied to perform the rehabilitation studies of the dam spillway with non-traditional design [94, 95]. A physical scale model of the existing spillway was first built and tested to enable engineers to determine the discharge capacity of the spillway and to collect data for verification of the numerical model. The numerical model was developed using experimental data from the physical model. It was then used to test several spillway modification alternatives, and to extend the analysis for cases which were not possible to simulate with the available experimental facility. The selected modification scheme was then finalized using the physical model. It shows that the composite hydraulic modeling strategy enables engineers to combine the advantages of physical and numerical modeling approaches to carry out this task. It not only reduces the design and analysis costs but also improves the quality of the results.

The subject spillway structure, as shown in Figure D-31 (a), has three 12-m wide gated bays on the top of a broad-crested weir of a concrete gravity dam located near the left embankment. The gates discharge water into a slopping chute with decreasing width followed by a steep slope section toward the curved downstream reach.

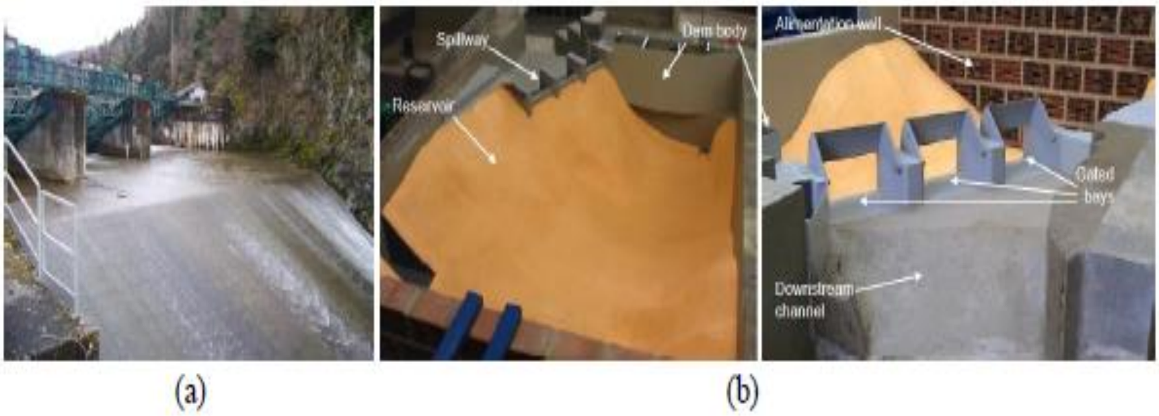


Figure D- 31(a) The existing spillway structure, and (b) physical model of the original spillway.

In addition to the non-conventional profile of the existing spillway, the unusual topographic condition of the downstream channel also made the study more complicated. Depending on the discharge, the flow control section appeared to be shifting. It may shift from the crest to a section farther downstream in the channel. Because of these unusual flow characteristics, a physical model study was considered as the most reliable approach to solve the problem. However, as the flow conditions were complex with increased discharges and the structure geometry was complicated, more design alternatives were investigated to develop the design. Since a physical model only approach for this project was more costly and time consuming, a 2-D depth-averaged numerical model study was incorporated to develop the modification (Figure D-32).

The numerical analysis was conducted based on the two-dimensional depth-averaged flow simulation approach. Since the flow over the spillway and tail channel was mainly two-dimensional, a full 3-D modeling approach was not adopted. This approach reduced the model preparation effort and computer time.

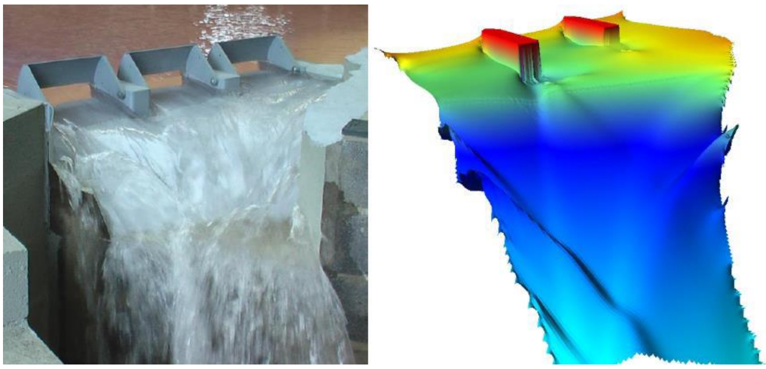


Figure D- 32 Physical (left) and CFD (right) modeling of the original spillway discharging 673 m³/s.

In all the simulations, it was assumed the gates were fully opened and discharging 673 m³/s design flow. The upstream boundary condition was the reservoir at the upstream side the spillway weirs. No downstream boundary condition was needed since the flow over the downstream steep channel is supercritical. The physical model (Figure D-33) was used to determine the discharge capacity of the original spillway. It was observed that the discharge coefficient of the original weirs decreases with increasing reservoir level until it approached a constant value of about 0.36.

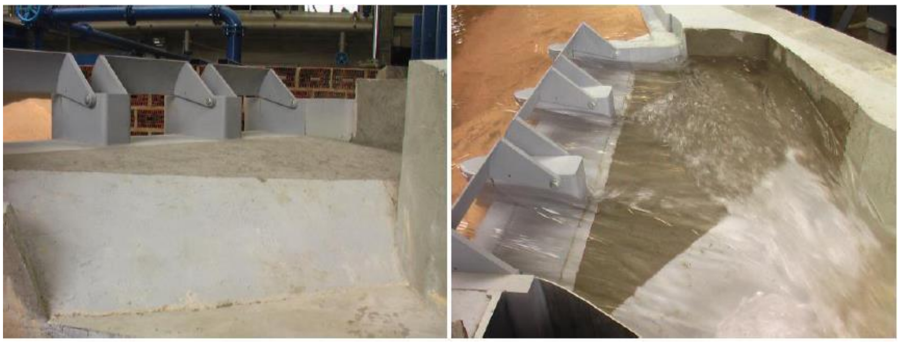


Figure D- 33 Physical model of the optimized spillway (left) and the model operating with the maximum reservoir level (right).

The observed low value of the discharge coefficient was probably caused by some or all the bays being submerged at higher discharges, with a shift of the control section from the weirs crest to the downstream channel whose width is narrower than the overall width of bays. It was observed that for a bay to operate properly, and thus to act as a control section of the flow, the Froude number should be increase from values below 1 to values above 1 near the weir crest.

Rattle Hydroelectric Project spillway model study

The 850-MW Rattle Hydroelectric Plant is a run-of-the-river power generation facility located on the Chenab River, downstream of the village of Rattle, near Drabshalla in northwest region of India near Kashmir. The Rattle HEP project consists of five orifice spillway bays and an upper level Ogee overflow spillway. It was observed in the early physical model studies that the flow trajectory from (a) bay no. 4 of the orifice spillway and (b) upper level spillway, strikes the left bank of the river. This could lead to slope failure of the left bank, in the plunge pool area and cause maintenance and operational problems. In order to improve the hydraulic performance of the spillway for future operation, modifications in the spillway design were developed to direct the flow trajectory away from the left bank. This was done by gradually increasing the width of (a) divide wall between spillway bay no. 5 and upper spillway and, (b) left abutment wall, up to the end of bucket to guide the flow nappe away from the left bank. As a design alteration, increasing the width of the abutment wall alone would have increased the risk of cavitation damage due to the resulting high discharge intensity. Therefore, a rotation in the lip of the bucket of the upper spillway was also investigated in iterative simulations for various combinations of angle of rotation and the gradually increased width of the divide wall and left abutment, near the lip.

Model study of spillways has been successfully carried out in India for more than fifty years at various research institutions. Numerical simulation on the other hand is often used as a complementary tool for hydraulic design and performance investigation. In spite of the advantage of high performance computers that can minimize effort, cost, and time, its accuracy can be impacted by improper selection of solver, solution parameters and boundary conditions.

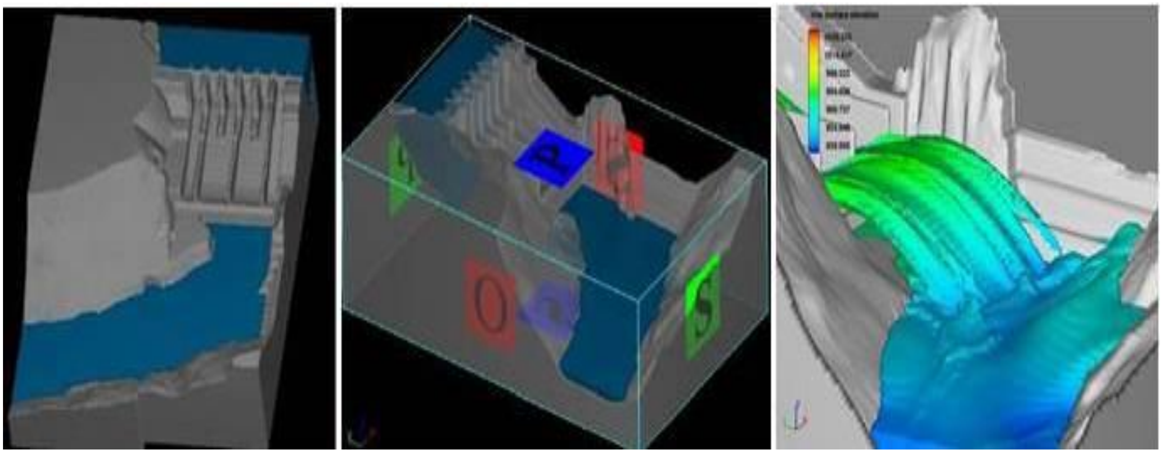


Figure D- 34(a) A view of the Rattle spillway 3-D simulation model; (b) boundary conditions for the 3-D CFD model; (c) a simulated plunging flow in the plunge-pool.

In order to develop a hydraulically sound modification of the spillway, iterative simulations of the flow over the spillway, for various modification alternatives were carried out to obtain a hydraulic design, compatible with the downstream topography. Model studies based on a 1:55 scale model were performed at the Irrigation Research Institute, Roorkee, which revealed that the trajectory formed by the upper level spillway and a orifice spillway strikes the left bank of the river. To avoid slope stability issues, modifications to the divide wall, left abutment and lip of the upper spillway were first developed with successive numerical simulations (Figures D-34a,b,c) using the Flow-3D CFD software package. The resulting design was then tested on physical model, thereby substantially reducing time and cost. The Flow-3D model simulates turbulence with a time averaged "Reynold's Averaged Navier-Stokes equations" turbulence model. The model requires a maximum turbulent mixing length, which represents the upper limit of the size of an expected turbulent eddy in the simulation and is used to limit the amount of turbulent energy created by the turbulence model.

The Flow-3D numerical model of the spillway was created based on a coarse discretization mesh having 975,000 active cells (uniform rectangular mesh of size 2m); to avoid prohibitive computer times and cost that would have resulted from a finer mesh covering all the bays with more details. The data so obtained was compared with that obtained from the physical model study and the model was verified, the subsequent simulations involving only the upper spillway and the orifice spillway bays no. 4 and 5, were performed using a model with finer mesh (uniform mesh of size 1m), covering only the desired regions in its domain. An equivalent uniform roughness of for the reservoir was computed based on Manning's "n" as 0.04 for banks and 0.035 for the bed. Considering the fact that the objective of the simulation was least sensitive to the roughness parameter, a value of 1mm for concrete surfaces and 60 cm for the reservoir was chosen as a reasonable estimate.

Setting the appropriate boundary condition can have a significant impact upon the extent to which the simulation results reflect the actual situation. The upstream boundary at 55m from the dam-axis has been specified as a pressure (stagnation) boundary (Figure D-34b) with a specified water level corresponding to the case of model testing that is being simulated. In the physical model, water levels for different values of discharge were recorded at this location, although the reservoir extended up to 1000m upstream of dam-axis. Thus a major approximation which could affect the discharge rates was introduced due to prohibitive computational effort that would have been required to model the reservoir conditions similar to the physical model. The downstream boundary condition was specified as an outlet boundary. All the other boundaries of the domain, except the top boundary which has been set at atmospheric pressure have been set to symmetry boundary condition, which minimizes computational effort as compared to a wall boundary condition. The domain on the downstream of dam was initially kept filled with water up to an elevation of 940 m.a.s.l. to allow approaching steady-state of flow condition. The model reservoir was filled in each simulation up to the water level recorded in the physical model corresponding to different discharges. The absence of rough surface condition visible in the trajectory in the numerical simulations could be ignored as it could have only been simulated on a mesh of size as small as 0.1m. Thus the numerical model was considered suitable for the purpose of recording water trajectories corresponding to different modifications done to the spillway design (Figures D-34b and D-34c).

The Flow-3D numerical model was verified by simulating different conditions of spillway operation that were also run in the physical model, and then comparing the discharges, velocities, the maximum level and the profile of the flow trajectory with the data obtained from model studies. Satisfactory comparison among the discharge values obtained from physical model studies, numerical simulation and empirical equations was obtained. All the simulations were run for a minimum run-time of 500 seconds to ensure that the flow has reached a steady-state. This was also confirmed by observing steady values of mean kinetic energy and volume of fluid in the domain. Minor errors were anticipated in the discharge values which could be attributed to the coarse discretization mesh used and the approximation of the upstream boundary condition. The boiling action of water in the plunge pool was well simulated with water levels rising up to a maximum of El. 969 m a.s.l (Figure D-35), compared to 970m model studies. The trajectory of water hitting the left bank of river was very well simulated the physical model studies, and could easily be verified by a comparison with the photographs available from model tests. One of the design modifications included directing the trajectory away from the left bank. Initial simulations involved a gradual increase in the width of (a) the divide wall between spillway bay no. 5 and upper spillway and (b) the left abutment, from the point where the width of piers chamfers in the original design (Figure D-36).

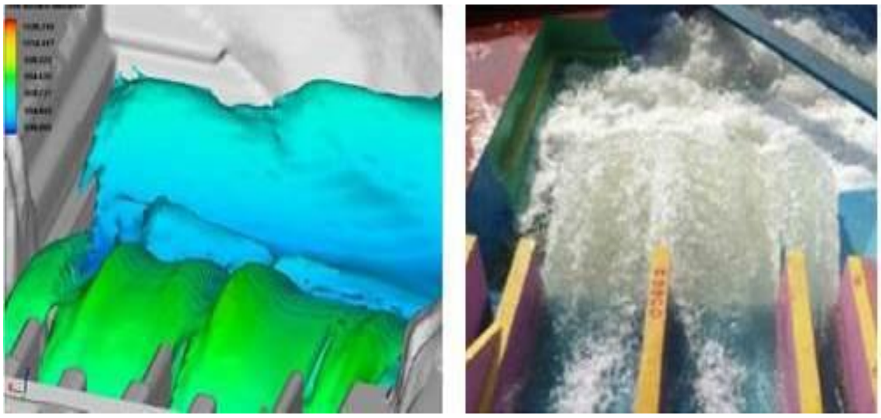


Figure D- 35 Simulated trajectory of water hitting the left bank in: (a) numerical simulation, and (b) physical model.

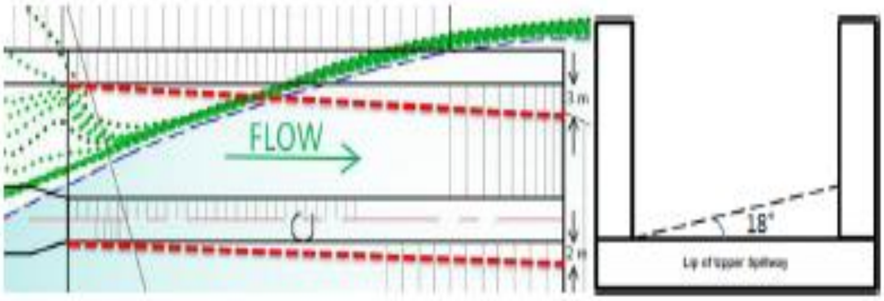


Figure D- 36 The width of the left abutment and the left-most divide wall was increased by 3m and 2m, respectively. A super-elevation of 18 degrees was provided at the lip of the upper spillway (right).

Since it was also needed to limit the resulting increase in discharge intensities, the lip of the upper spillway was rotated along an axis perpendicular to the dam-axis and passing through the right corner of the lip, as an additional design alteration. A number of simulation runs were made with different combinations of rotation angle of the lip and widths of the divide wall and the left abutment. One of the CFD runs is depicted in Figure D-31. A rotation of 18 degrees (Figure D-30) along with an increase of 3m and 2m in the width of left abutment and divide wall respectively, were found to be the minimum design alterations to sufficiently deflect the trajectory away from the left bank of the river.

Conclusions.

The results obtained from the numerical simulations provided sufficient confidence to the engineers to proceed further with physical model testing of the spillway with the final modifications evolved from iterative simulations. Although, the use of a coarse mesh reduced accuracy in the discharge values, the trajectory of water was simulated quite accurately by Flow-3D, (see Figure D-35 to compare results of model testing and numerical simulation). Figure D-37shows how the trajectory of water coming out from the spillway, which hit the left bank in the plunge-pool area, got deflected away from the bank by making design modifications. The CFD model was successfully applied to aid the model testing of spillway for design alternatives [113].

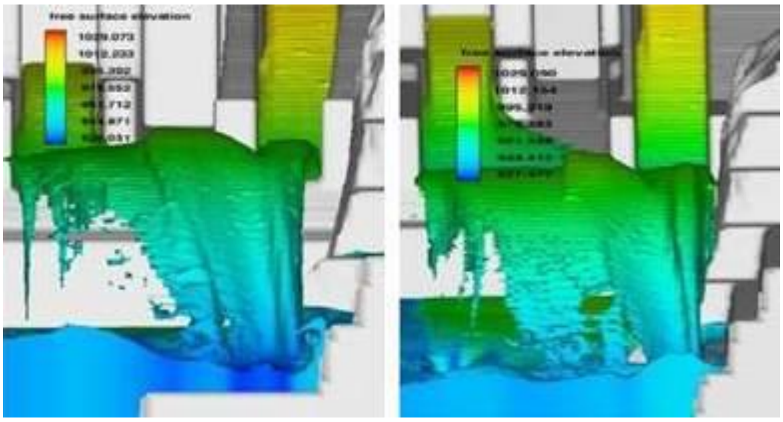


Figure D- 37 Front view of simulated flow trajectory by Flow-3D: (a) with modified divider-wall, left abutment and spillway lip and (b) without design modifications.

Boundary Dam Spillway Modification Project Model Studies

The Boundary Dam (Figures D-38 to D-41) is located on the Pend Oreille River in northeastern Washington, USA. The project consists of a 340 ft. high concrete arch dam, seven low level sluiceway outlets, two high level overflow spillways (Spillways no. 1 and no. 2), and a 1,003 MW power plant. The spillway and sluiceway discharges produce high Total Dissolved Gas (TDG) concentrations in the tailwater and the downstream reach. Studies were conducted to develop modifications to the spillways to mitigate this gas production issue. Based on both physical and numerical hydraulic model analysis, the proposed modifications to both spillways were tested and finalized. For the Spillway no. 2, the modifications involved installation of nine baffle blocks with an upstream ramp structure that provides external air to the flow over and around the blocks. The ramps and blocks create an efficient breakup of the chute jet prior to its drop into the plunge pool, thereby helping to limit overall gas transfer. The hydraulic design of the spillway ramp and baffle blocks required extensive hydraulic analysis using both physical and numerical modeling. The physical model coupled with the numerical modeling and field observations allowed the assessment of the TDG performance for a wide range of flow conditions and baffle block and ramp arrangements.

Several model studies were conducted to gain understanding of the hydrodynamics of the gas exchange processes in the tailrace of the original operational conditions, and to develop modification alternatives of the spillway for mitigation of the TDG issues. The CFD numerical models, which were developed based on the Flow-3D software package, were used in conjunction with physical scale models. This approach took advantage of the relative strength of the two modeling methods. The physical model was used to its greatest utility in iteratively developing geometry changes of the spillway modification alternatives, while the CFD models with prototype-scales were used to provide quantified information of hydrodynamic loads for structural design and flow field data for predictions of the TDG levels.

Figure D-38 Spillways in operation.

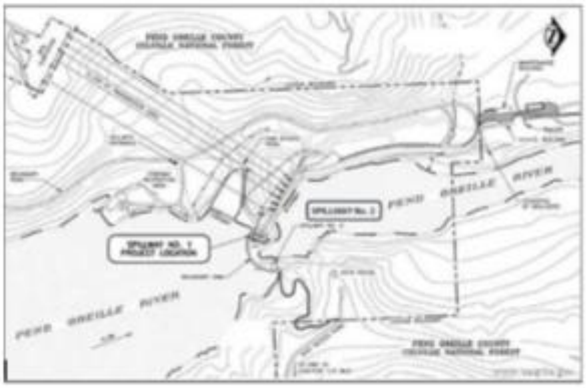


Figure D- 39 Boundary dam key features.

Figure D- 40 Downstream view of Boundary Dam

Figure D- 41 Spillway no.2 with roughness elements.

The physical model studies were carried out based on a 1:25 scale model which was used to: develop spillway discharge rating curves; provide velocity measurements for the numerical model calibration; characterize the plunge pool performance; investigate spillway performance with modifications; and optimize the design of the roughness block-elements. Although this physical model cannot replicate the prototype gas transfer process and cannot directly predict TDG performance, it provided insight for an understanding of the hydrodynamics of spillway discharges into the plunge pool and the tailwater that influence the gas exchange processes. The model provides direct visualization of the flow patterns generated in the tailrace and the relative depth to which entrained air bubbles are driven by the flow. It also provided insight into how project operations and spillway modifications might be developed to mitigate the TDG production.

The numerical model was developed based on the Flow-3D software package of the Flow Science Inc.

It is a full 3-D CFD fluid dynamics simulation software which is capable of simulating many hydrodynamic phenomena especially free-surface flows. Its pre- and post-processing modules with advanced data visualization capabilities make its applications easier. Full 3-D CFD models of spillways no.1 and no.2, a sluiceway, and the plunge pool and tail channel were developed for the spillway modifications study to gain understanding of the governing hydraulic and hydrodynamic processes driving gas exchange in the tailrace. These CFD models were used in conjunction with the physical model to provide quantified information such as hydrodynamic loads for design and flow fields for prediction of the TDG levels. The models were verified and validated based on the physical model and field observation data. The 3-D simulations of the unmodified and modified spillway no.1 discharging 10,000 cfs and 13,000 cfs are depicted using the visualization module and shown in Figures D-42 and D-43 respectively. Figure D-44 shows CFD simulated operations of unmodified and modified Spillway no.2.

The CFD models were found very useful and its contributions include: analysis of ramp and baffle blocks arrangement for reducing TDG levels; estimation of hydraulic loads on the roughness elements; investigation of the effects of spillway modifications on the cavitation potential of the spillway floor and the tailrace flow fields; and generation of hydrodynamic input for prediction of TDG levels.

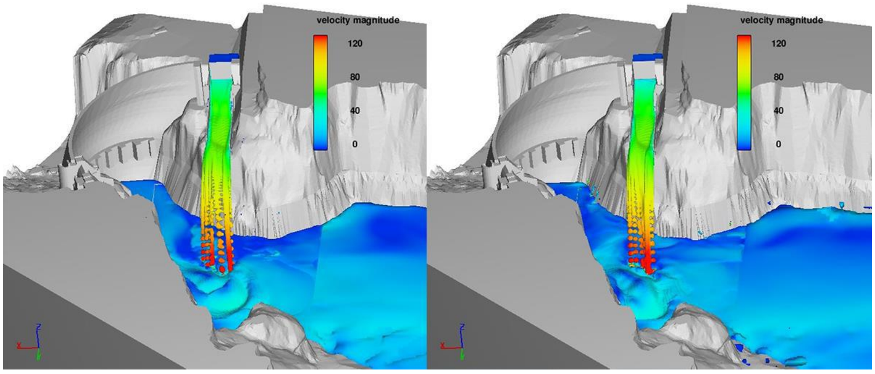


Figure D- 42 CFD simulation of the unmodified spillway no.1 passing 10,000 cfs (left), and 13,000 cfs (right).

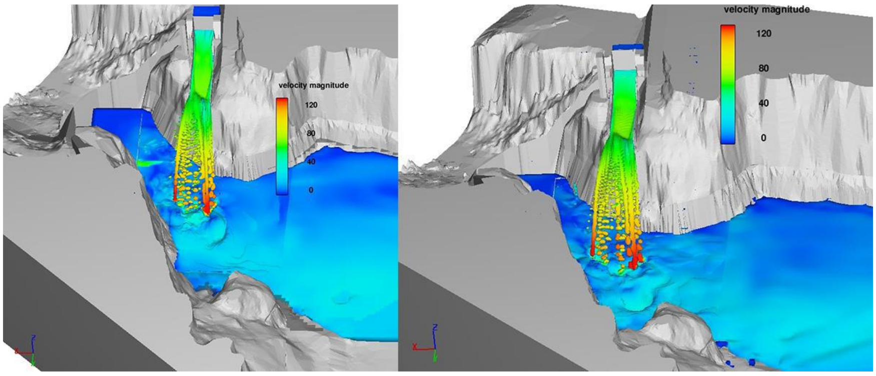


Figure D- 43 CFD simulation of the modified spillway no. 1 passing 10,000 cfs (left), and 13,000 cfs (right).

In summary, a physical model coupled with numerical models and field observations were applied to develop the hydraulic design and to allow analysis of Total Dissolved Gas performance. The resolution of many hydraulic design issues relied heavily on the results of both models. Both models were used successfully in a complementary manner for maximum contribution of their respective strength [114-116].

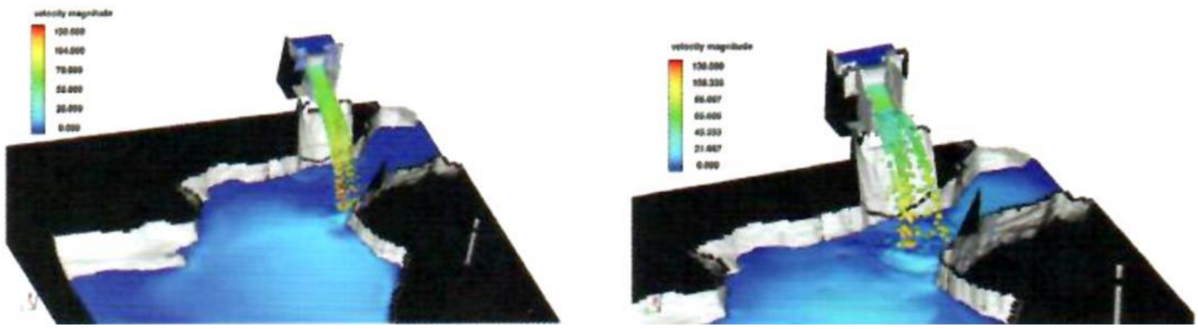


Figure D- 44CFD simulated operation of unmodified (left) and modified (right) spillway no.2.

8. Mathematical and Numerical Modeling

To solve a hydraulic problem a set of algebraic and differential equations that represents the interaction between the flow and process variables in space and time is developed first as the mathematical model of the problem. It is based on certain set of assumptions about the physics of the prototype flow and associated environmental processes. These assumptions will set clear limits to the domain of applicability of the mathematical model and any computational and numerical model that may be derive from it. A prerequisite for the development of a mathematical model is an understanding of the key physical processes involved, leading either to fundamental principles such as Newton‘s laws of motion or to well-established empirical relationships such as Chezy and Manning resistance laws.

Mathematical models of most physical phenomena are non-linear, necessitating the need to develop approximate solutions through numerical methods with the aid of electronic computers. Hence, a numerical model is an approximation of a mathematical model of some prototype situation, giving a computable set of parameters that describes the flow at a set of discrete points. Many numerical models can be formulated from the same underlying mathematical model by employing alternative numerical methods and mathematical manipulations. The performance of the numerical models will be determined by the properties of the numerical methods employed, and for the same geometric and boundary data may give significantly different results. These differences are often masked, in part, by the calibration procedure. A numerical model, like a mathematical model, is not specific to any particular site, and the strength of both these types of model lies in their generality. A specific application will require data from the prototype site and a computer with a program to organize the data and execute the calculations [32, 66, 69].

A computational model is an implementation of a numerical model on a computer system with the relevant data from a specific site. The results of the computational model depend on a variety of factors, including the quality of the prototype data, the details of data processing, possibly the internal organization of the calculations and the type of computer used. Many computational modeling systems and packages are available for a variety of hydraulic engineering problems. The end user may have to choose which model to use, and certainly will have to be able to interpret the model results critically and responsibly. The USACE Modeling Mapping and Consequences Production Center (MMC) provide hydraulic modeling, mapping and consequence analysis for USACE levees in support of the USACE Levee Safety Program. The MMC also provides Flood Inundation Modeling support during real-time flood events with its Flood Inundation Modeling Cadre (FIM). The MMC has developed processes, tools and standards for creating levee breach hydraulic models for use in emergency preparedness plans (EPP), during real-time flood events, and in support of the Corps Levee Safety programs. The MMC-developed standards have been used to provide levee failure modeling for several USACE levees and multiple flood events throughout the continental U.S. As an example, the MMC used the two-dimensional capabilities within the Hydrologic Engineering Center‘s River Analysis System (HEC-RAS) to perform a quick Levee Break analysis during a real-time flood event [135-137,152].

9. Numerical Methods of Solutions

Hydraulic simulation models have been applied to address concerns include: hydraulic structure and waterway design/operation; effects of navigation traffic on waterways; water and environmental quality management; contaminated groundwater resources; wetland maintenance and management; flood-control; channel design and operation; overland, bank and near-structure erosion; and watershed runoff and flow analysis etc. These modeling tasks include creation and/or enhancement of computational and process understanding within the models, and improved modeling productivity through the coupling of these models with comprehensive and consistent graphic user interface, visualization, and parameter estimation methods. Linkages to Geographic Information System (GIS), optimization and uncertainty methods, and decision support techniques are also key components of these system developments. The developed modeling systems can increase the efficiency and efficacy of modeling to both modeling specialists and decision-makers/managers. One-dimensional hydraulic computer modeling is commonly used for river system studies and floodplain mapping, but there are instances when one-dimensional modeling is not appropriate to model flood flows. In instances where flood flows overtop the channel banks and the floodplain is flat, two-dimensional hydraulic modeling may be required.

By neglecting different less-significant terms of the governing equations depending on their magnitude relative to the other terms, several analytical procedures have been developed to analyze unsteady flows. However, if all terms must be retained in the analysis, then numerical methods must be used. The most widely used methods include: (1) Finite-difference methods; (2) Method of characteristics; (3) Finite-element method; and (4) Volume-of-fluid method etc. In the finite-difference methods, the partial derivatives are replaced by finite-difference quotients, and the resulting algebraic equations are then solved to determine the flow conditions. In the method of characteristics, the governing equations are first converted into characteristic form, which are then solved by a finite-difference scheme. In the finite-element method, the flow system is divided into a number of elements (or sub-domains), and the partial differential equations are integrated at the nodal points of the elements [78]. The capabilities of advanced CFD (Computational Fluid Dynamics) modeling for analyzing complex hydraulic structures has been demonstrated by many researchers and engineers. Some example exhibits of CFD model applications are shown in the following figures. (Figure D-45 to D-52).

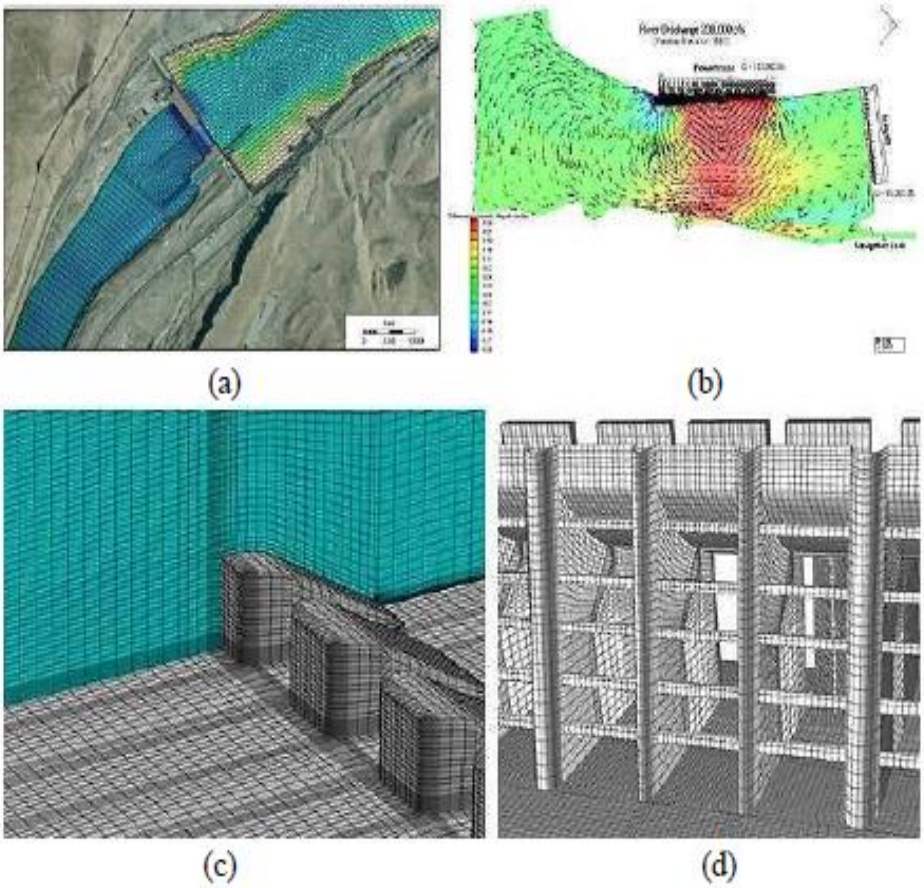


Figure D- 45 (a) A CFD model of a dam project showing the bathymetry and computational mesh near the dam, (b) A 2D CFD simulation of flow pattern downstream of a project, (c). An example computational mesh in the stilling basin and near the baffle blocks of a spillway, and (d) Turbine intake mesh of a CFD model.

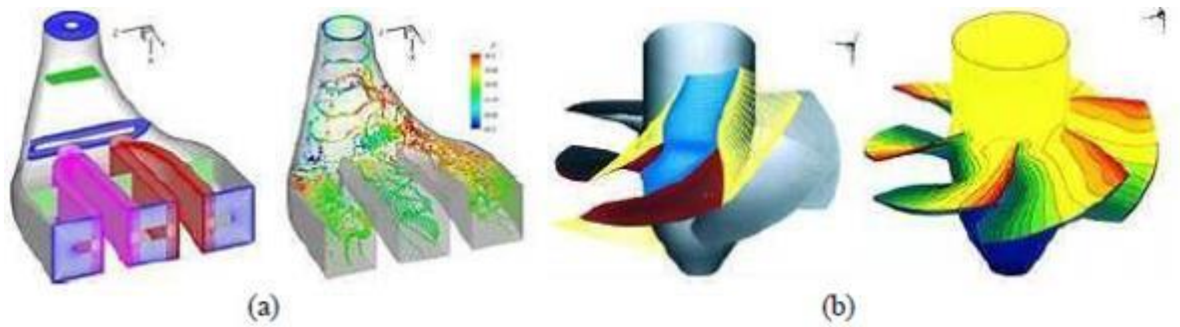


Figure D- 46 (a).Simulation of flow through a draft-tube. (b) Simulation of flow through a Kaplan turbine. (SAFL)

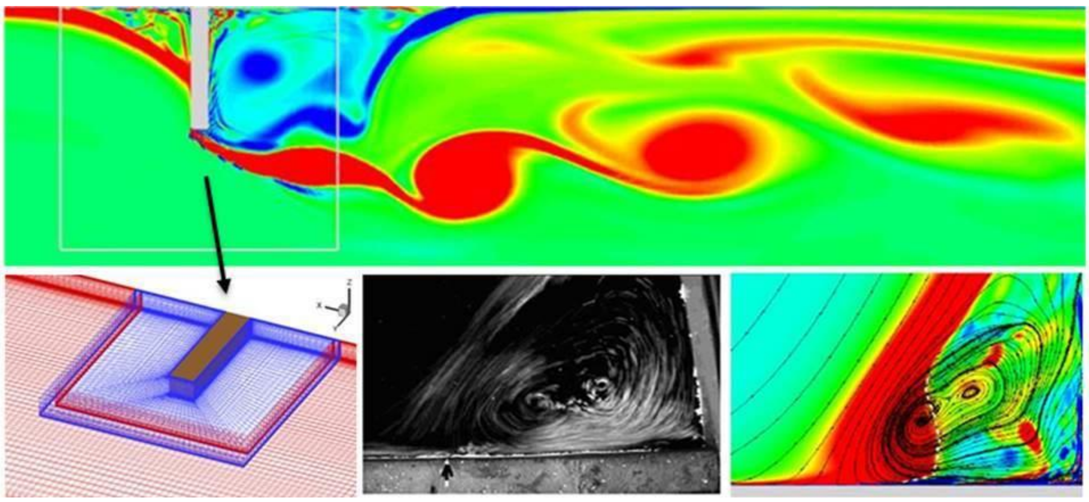


Figure D- 47 CFD modeling of unsteady flow passing a bridge abutment. (SAFL)

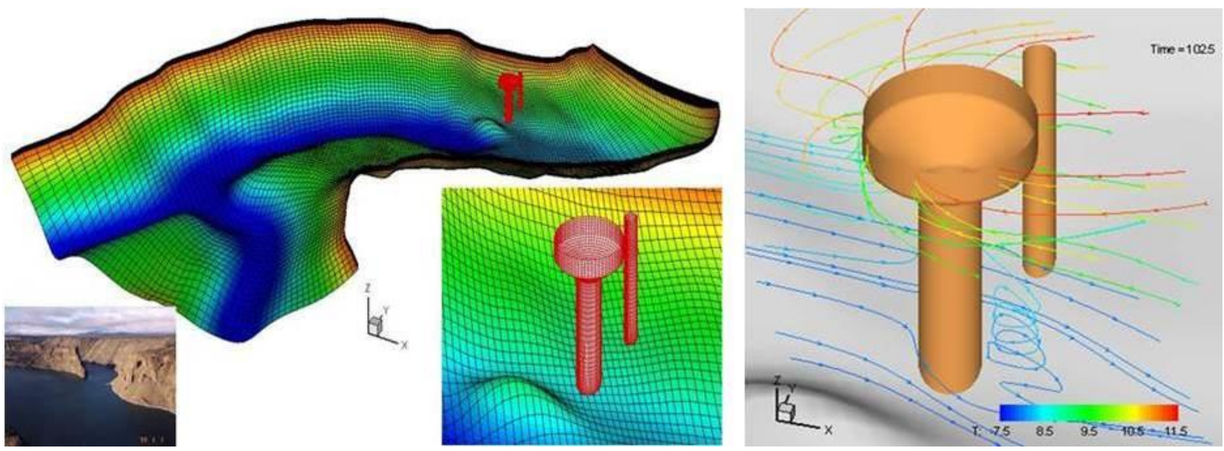


Figure D- 48 CFD simulation of flow toward an intake structure (SAFL)

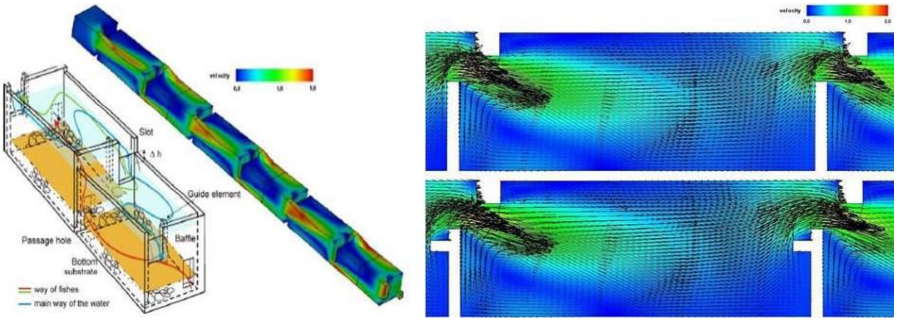
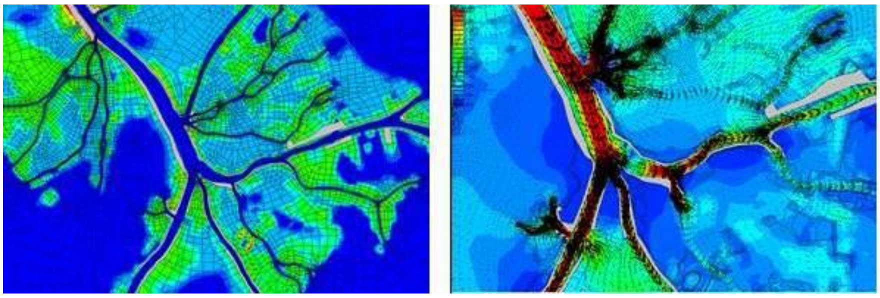
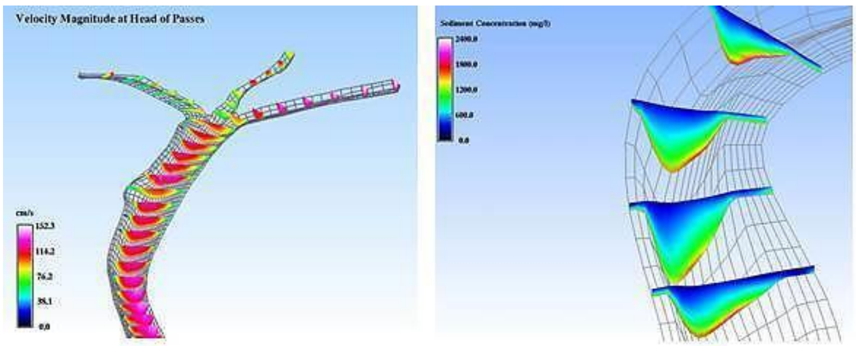


Figure D- 49 CFD simulation of the flow through a pool-slot fish way [67].



(a)



(b)

Figure D- 50 (a) Mississippi River Delta Model Mesh and simulated flow, (b) 3D simulation of flow pattern and sediment loads in a river reach

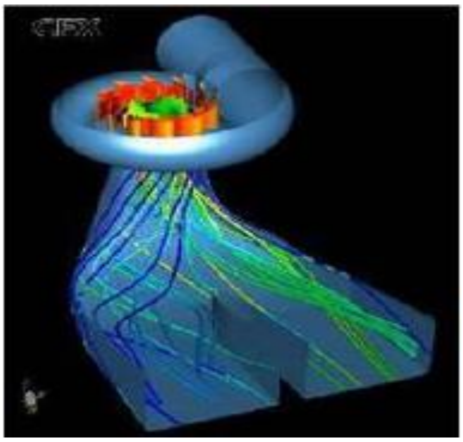


Figure D- 51 CFD simulation of flow through a Francis turbine

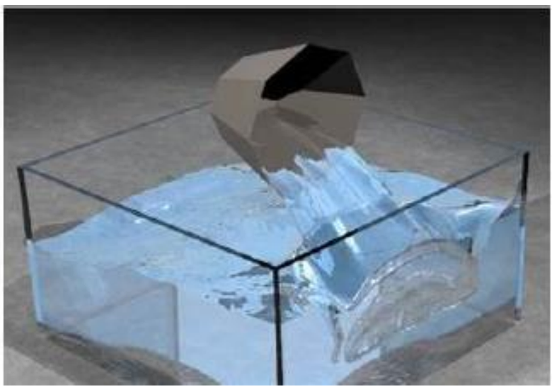


Figure D- 52 Pouring water into a container.

Several commonly used numerical modeling techniques in hydraulic engineering are briefly described as follows. It includes the finite difference methods, the method of characteristics, the finite element methods, and the volume-of-fluid method.

10. Numerical Modeling based on Finite Difference Methods

The Finite difference method (FDM) converts a linear ordinary differential equations (ODE) or non-linear partial differential equations (PDE) into a system of equations that can be solved by matrix algebra techniques. To numerically integrate the ordinary or partial differential equations, the derivatives terms are replaced by finite-difference approximations through Taylor series expansion of the functions and resulted in a set of algebraic equations that makes the problem of obtaining the solution to a given ODE/PDE ideally suited to modern computers [125].

To use a finite difference method to obtain approximate solution of a problem, one must first discretize the problem's domain. This is usually done by dividing the domain into a uniform grid (Figure D-53). This means that finite-difference method produces sets of discrete numerical approximations to the derivative, often in a "time-stepping" manner. The quality and duration of simulated FDM solution depends on the discretization equation selection and the time and space step-sizes. The data quality and computational time increase significantly with smaller step size. Therefore, a reasonable balance between data quality and simulation duration is necessary for practical applications.

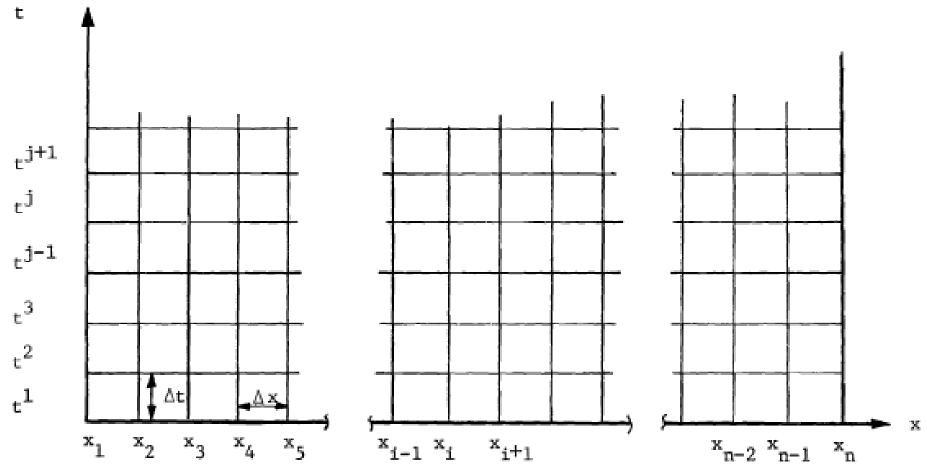


Figure D- 53 Finite difference grid network in the x-t plane.

Large time steps are useful for increasing simulation speed in practice. However, time steps which are too large may create instabilities and affect the accuracy of the results. There are several techniques available to solve the governing equations numerically by finite difference such as central differencing, forward differencing, explicit method, implicit method and Crank–Nicolson method etc. In the Crank-Nicolson method, the central difference at time and a second-order central difference for the space derivative are applied. This scheme is numerically stable and convergent but usually more numerically intensive [125, 126].

Solving one-dimensional unsteady open-channel flows by finite difference method. The continuity and dynamic equations describing the one-dimensional unsteady open-channel flows are commonly referred to as Saint-Venant equations. Mathematically, these equations are a pair of first-order partial differential equations of hyperbolic type. No analytical solutions are known for these equations except for a few very special cases for which the equations are considerably simplified and often with linearization to allow solutions. In general, they are solved numerically with differential terms approximately by finite differences of selected grid points on a time-and-space domain. Substitution of the finite differences into a partial differential equation transforms it into an algebraic equation. Thus, the original set of differential equations can be transformed into a set of simultaneous discretized algebraic equations for solution. Flood routings are generally carried out by using the finite difference numerical schemes [75, 125].

The governing unsteady, one-dimensional Saint-Venant equations of open-channel flows are commonly solved by a finite difference scheme (often by an implicit scheme) to handle general channel and river flows. The initial conditions for the unsteady flow are provided by solving the steady varied flow equation for the specified boundary conditions. The solution for the unsteady flow allows many separate boundary conditions to be specified which are composed of combinations of specifying the depth or discharge as functions of time at either the upstream or downstream ends, with the stage-discharge relation or constant depth and flow rate specified at the other end. Typical solutions show the spatial and time dependency of such flow characteristics as flow rate, depth and velocity. It also includes lateral inflow, and channels whose geometry, slope, and Manning's n vary with respective to distance along the channel.

Solving flow problems by inverse finite difference solution method.

One notable method of solving flow problems with finite different methods is the re-formulation of the problem in an inverse space through inverse formulation. As an example, the problem of flow through a circular orifice is formulated by considering the velocity potential and Stokes's stream function as the independent variables and the radial and axial dimensions as the dependent variables, and a finite difference solution is obtained to the resulting boundary-value problem. This inverse formulation has the advantage over a finite difference solution in the physical plane that the region of flow is rectangular and consequently well adapted for minimum logic in programming a digital computer. The inverse finite difference solution is more readily obtained than a comparable solution in the physical plane, even though the inverse partial differential equation and associated boundary conditions are non-linear. The results from the inverse finite difference solution are in close agreement with other most recent results from approximate solutions to this problem. The inverse method of solution is applicable to other free streamline as well as confined axisymmetric potential flow problems. The essential difference in other problems will be in the boundary conditions.

The inverse finite difference solution method can also be applied to solve three-dimensional potential flow problems. A finite difference method is developed to solve the three-dimensional, steady, incompressible, potential flow equations obtained by using a velocity potential function, and two mutually orthogonal stream functions, to describe the flow. Problems are formulated in an inverse space where the potential function and the two stream functions are the independent variables, and the Cartesian coordinates x, y, and z are the dependent variables. The boundaries of the problem in the physical space, including the free surface, have known positions in the inverse space, so trial and error adjustments to the positions of the boundaries are unnecessary. Methods of describing the effect of the placement of a body, whose shape is partially specified, in the flow field can be developed using finite differences, and a solution for the x-, y-, and z-coordinates is obtained at each grid point formed by the intersection of surfaces held constant with respect to velocity potential and the two mutually orthogonal stream functions in the inverse space. The inverse finite difference solution method has been applied to solve the problem of steady-state partially saturated infiltration of moisture applied over a horizontal source circle which moves through homogeneous soils toward a water table. The inverse-formulation reverses the usual role of the variables and the solutions are obtained by using an iterative procedure. Subdivisions of the computational grid network are often needed to maintain the overall accuracy of the results and maintaining the manageable amount of computer time.

Subdivided grid network. An example of the grid network subdivision is shown in Figure D-54a and a flow net obtained by the inverse finite difference solution method depicting axisymmetric infiltration from a circular source toward a water table is shown in Figure D-54b.

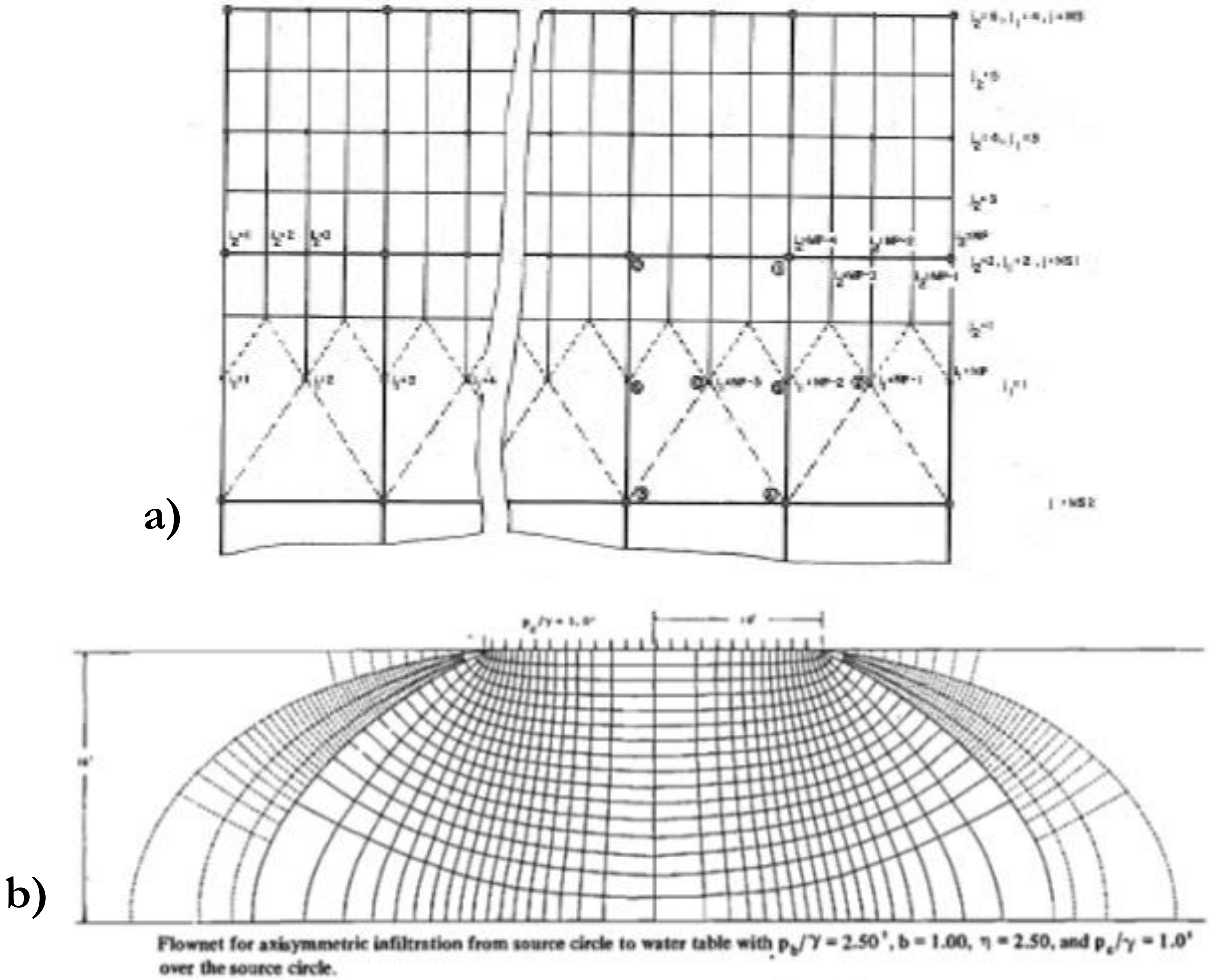


Figure D- 54(a).A finite-difference grid network with subdivisions; (b) Flow net by the inverse finite difference for axisymmetric infiltration.

Three-dimensional finite-difference modeling.

A 3-D finite-difference model was developed to investigate the flow conditions downstream Keenleyside Dam on the Columbia River. A partial view of the finite-difference mesh and a comparison of the simulated flow pattern with measurements using a Doppler current meter are shown in Figure D-55 [127].

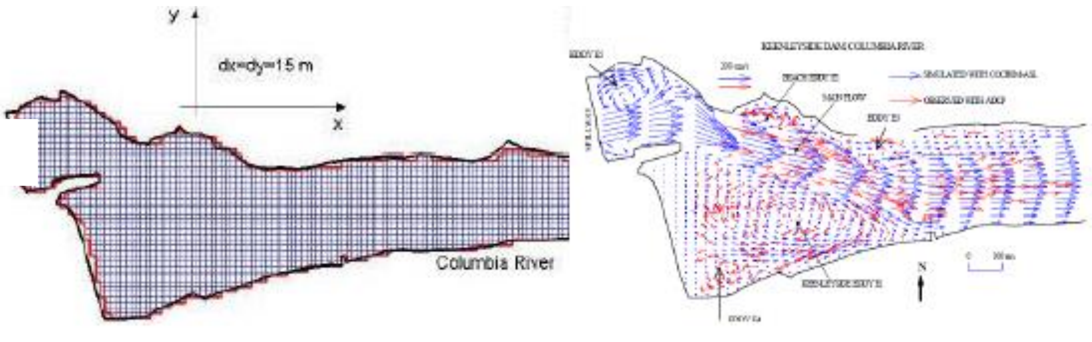


Figure D- 55 A partial Finite-difference mesh (left) and the simulated and observed flow pattern.

Solving flow problems by finite difference method with boundary-fitted grid network.

Many flow problems can be solved by finite difference methods using a boundary-fitted, non-orthogonal grid in the horizontal plane. This modeling method simulates unsteady free-surface three-dimensional hydrodynamics, constituent and sediment transport, and mobile-bed dynamics in natural waterways. To obtain the hydrodynamic solution, it first solves the depth-averaged Reynolds approximation of the momentum equations coupled with the depth-averaged mass conservation equation to yield the depth-averaged velocity and water-surface elevation on a two-dimensional grid. This solution is based on finite-difference approximations applied to a boundary-fitted, non-orthogonal grid in the horizontal plane. The deviations from the depth-averaged velocity are then computed for each computational cell through solution of the mass-conservation equation for vertical momentum diffusion on a vertical grid. The basic equations were solved via the finite-difference method. The finite-difference solution algorithm replaced continuous derivatives in the governing differential equations with ratios of discrete quantities. Quantities computed by the model included three-dimensional velocities, surface elevation, vertical viscosity and diffusivity, temperature, salinity, and density.

As an example of developing a computational grid, for the Chesapeake Bay study, the computational grid was developed by dividing the Bay into a grid of discrete cells. To achieve close conformance of the grid to Bay geometry, cells were represented in curvilinear rather than rectangular coordinates. A z-plane grid was employed in which the number of vertical layers varied depending on local depth. Velocities were computed on the boundaries between cells, and temperature, salinity, and density were computed at the center of each cell (Figure D-56) [127].

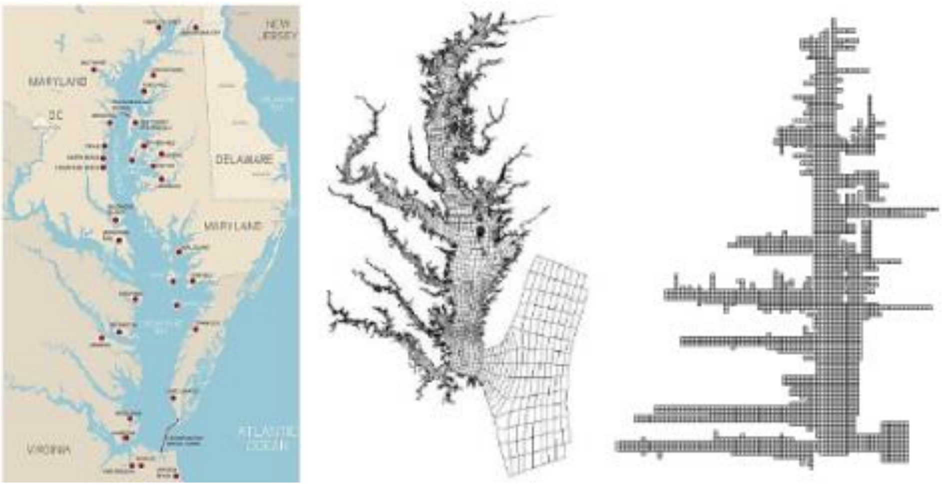


Figure D- 56 A map of the Chesapeake Bay, the computational grid network, and the grid in transformed coordinates of the Bay.

11. Numerical Modeling based on the Method of Characteristics

The problem of hydraulic transients (water-hammer) of an unsteady closed-conduit flow system can be solved by seeking solution of momentum and continuity equations. The most common method of solving these equations is the "method of characteristics." A general solution to the governing partial differential equations is not available; however, a numerical solution of the equations can be obtained.

The partial differential equations can be transformed by the method characteristics into particular total differential equations. These equations may then be integrated to yield finite difference equations such that a numerical solution can be obtained on a digital computer with the appropriate boundary and initial conditions incorporated. The equations in their simplest form, not including smaller minor terms, are first organized for numerical computations. The continuity and momentum equations form a pair of quasi-linear hyperbolic partial differential equations in terms of two dependent variables, velocity and head, and two independent variables, distance along the conduit and time as shown in Figure D-57. The equations are simplified to exclude the smaller minor terms and converted into four ordinary differential equations by the method of characteristics such that an orderly numerical solution can be obtained with prescribed time interval [77]. An illustrative example of numerically simulated hydraulic transients for a pumped-storage project is shown in Figure D-58.

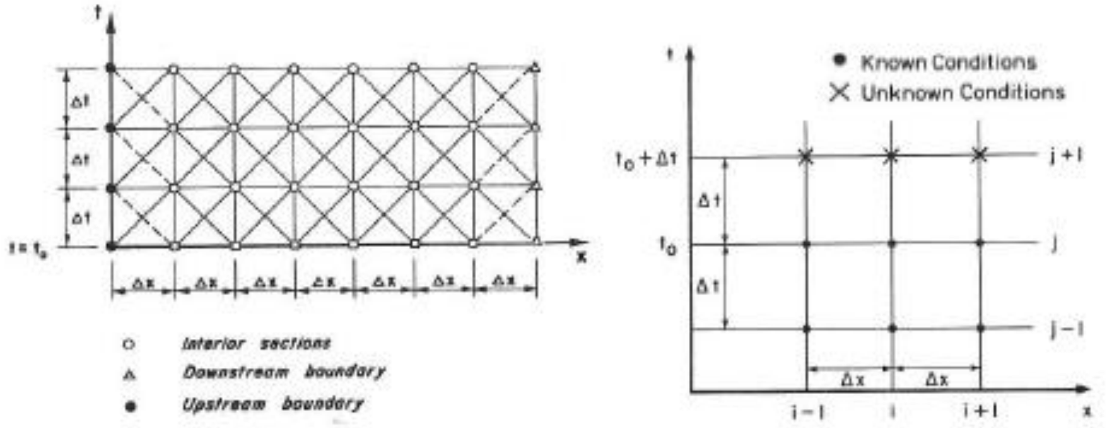
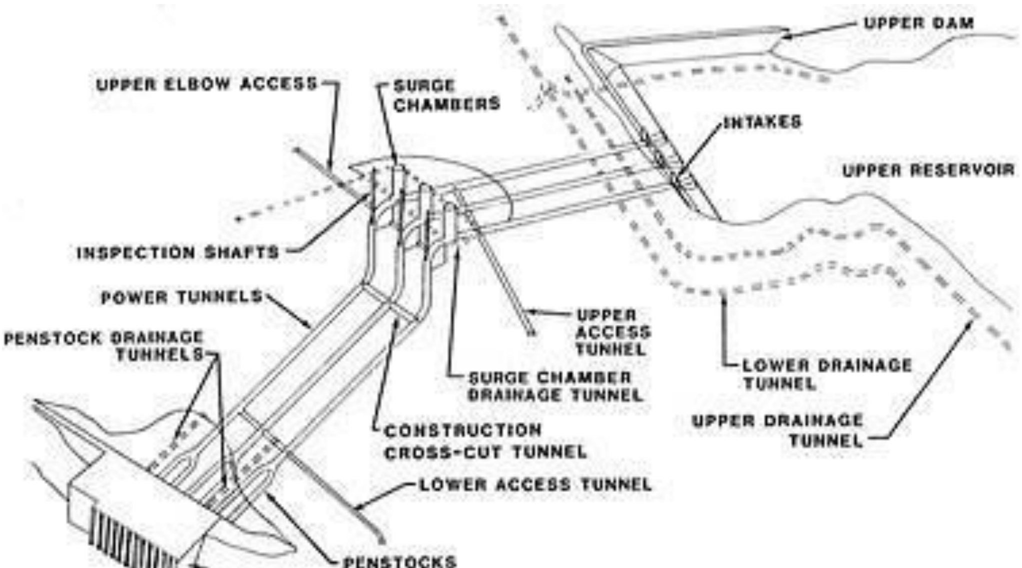


Figure D- 57 Computational grid for method of characteristics.



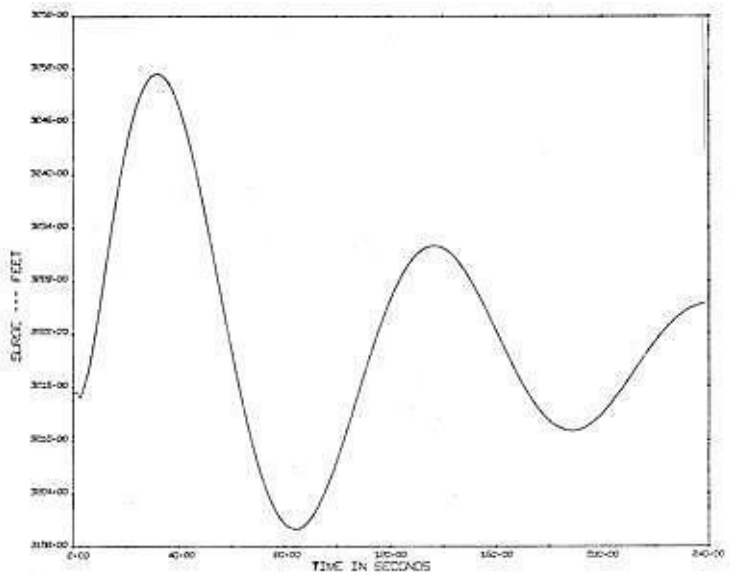


Figure D- 58 A typical hydraulic transients (water-hammer) analysis for a pumped storage project.

A study of the Jordan River power plant valve closure transients by the method of characteristics.

The Jordan River power plant is located in British Columbia, Canada. It is a peaking power plant. As shown in Figure D-59a, the upstream conduit consists of a tunnel having a 5,285-m-long, mainly D-shaped section; 82-m-long , 3.96-m-diameter, and 451-m long, 3.2-m diameter sections; and a 1400-m long penstock reducing in diameter from 3.2 to 2.7 m. There is only one Francis turbine rated at 145 MW and 265.5-m rated head. To reduce the maximum transient-state pressure, a pressure regulating valve (PRV) is provided. This study was carried out to determine the transient conditions caused by closing of PRV. The simulation was carried out to establish steady flow first by opening PRV from 0 to 20 percent at a very slow rate. The PRV was then closed from 20 to 0 percent to generate the transient conditions. The comparison of computed and measured results showing pressure changes are given in Figure D-59b [78].

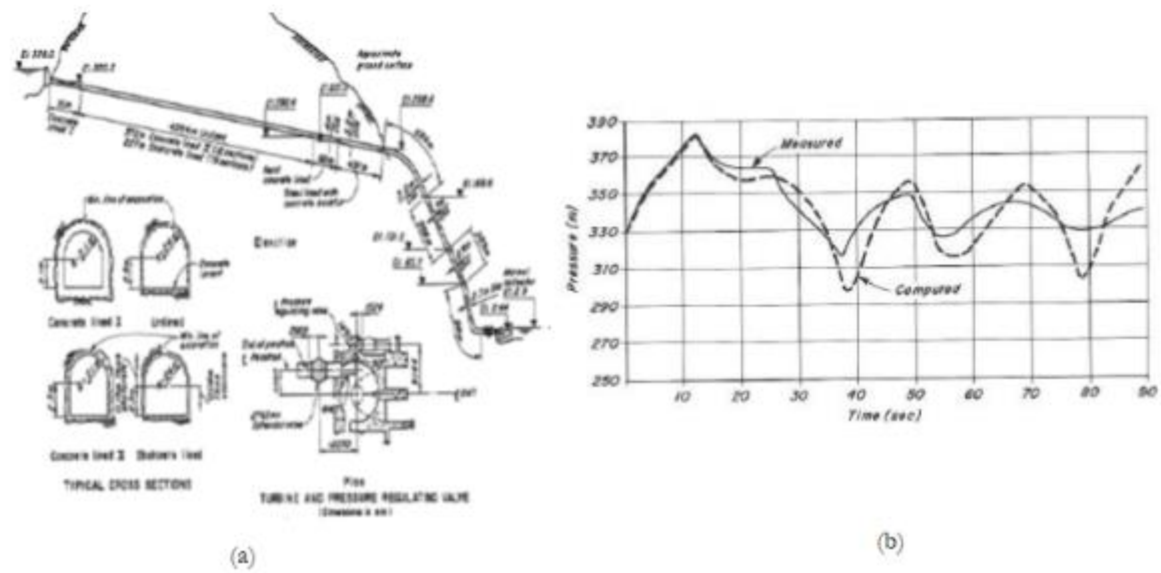


Figure D- 59(a).The profile of the Jordan power plant. (b) Comparison of computed and measured results showing pressure changes due to a valve closure.

12. Numerical Modeling based on the Finite Element Methods

Finite element method (FEM) is a numerical method for solving the governing differential equations describing the physics of a flow problem. The method treats the space- and time-dependent flow problems in terms of partial differential equations (PDE). For most flow problems, these PDEs cannot be solved with analytical methods except some simplified problems. Instead, an approximation of the equations is developed using different types of discretization methods. These discretization methods approximate the PDEs with equations that can be solved using numerical methods. The solution to the numerical model equations are an approximation of the real solution to the PDEs. In the finite-element method, a flow system is divided into a number of sub-domains called elements that forms a finite-element mesh, and the partial differential equations are integrated at the nodal points of the elements. The method essentially consists of assuming the piecewise continuous function for the solution and obtaining the parameters of the functions in a manner that reduces the error in the solution. To obtain a finite element solution, a finite element mesh representing the flow domain of complex shape or geometry is first created, and a system of equations is then assembled, for which the elemental matrices and vectors need to be evaluated. The resulting linear system of equations is solved by incorporating the associated boundary conditions. The time-dependent problems are solved by treating the derivative with time by the finite difference approximation. The derivatives with respect to spatial variables are handled in the usual way of FEM. A coarser mesh (larger subdivisions) results in less-accurate results, but a finer mesh, which creates more elements and requires more computing power to solve. That‘s why a mesh size that varies across the domain is useful; one can define a coarser mesh in areas that are of less importance, and a finer mesh in areas where good accuracy is required. The results can be presented in graphic forms through use of post-processing software for graphic presentations and visualization of the results [79, 80]. Examples of finite element meshes are shown in Figures D-60, D-61 and D-62.

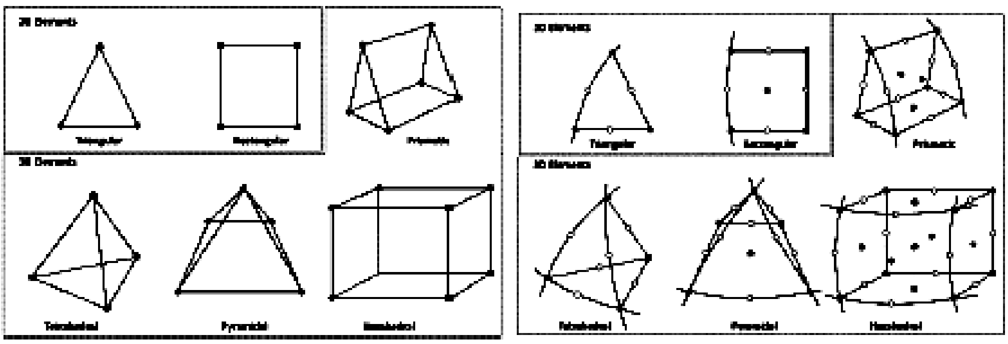


Figure D- 60 Example node placement and geometry for 2-D and 3-D linear elements (left) and second-order elements (right)

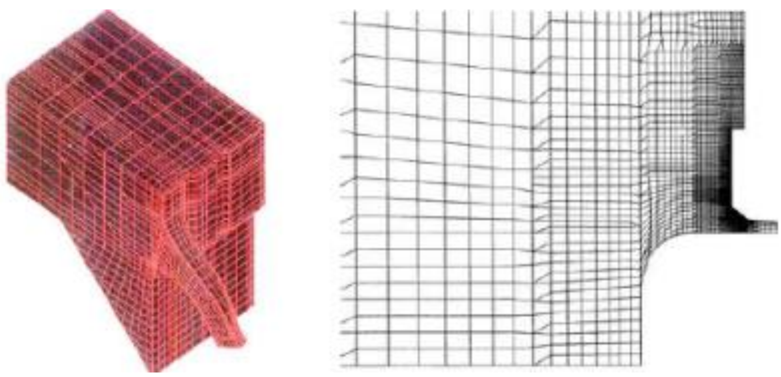


Figure D- 61 A 3-D finite-element model of flow approaching an intake (left) and a 2-D finite element model of flow through a gate opening (right).

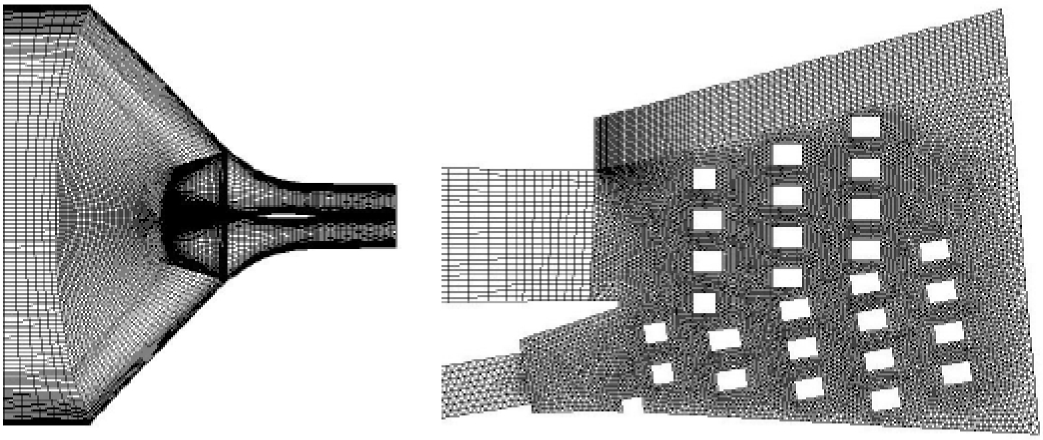


Figure D- 62 Finite-element mesh of the spillway basin and the downstream spillway chute.

An example of solving the two-dimensional free-jet problem with adjustable finite-element mesh is shown in Figure D-63. Good agreement between the finite element solutions (by an iteration procedure) and experimental data can be seen in Figure D-63c [101].

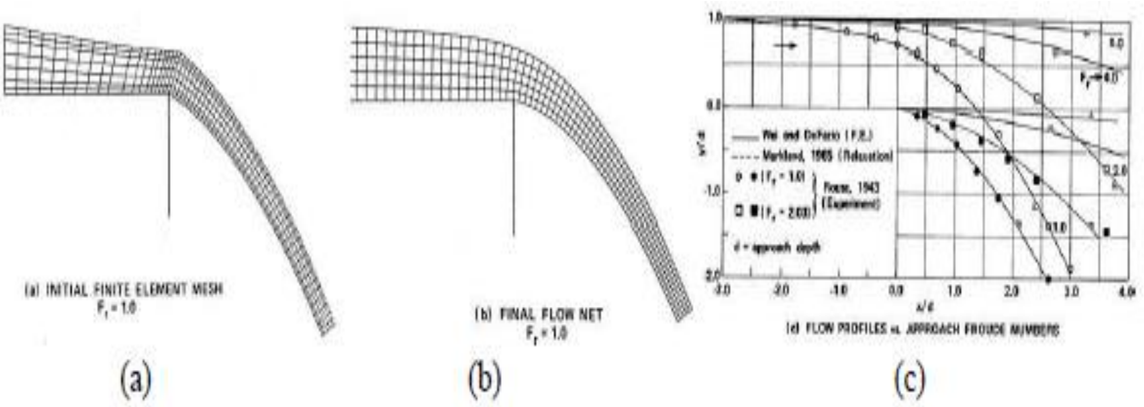


Figure D- 63 Simulation of free-jet trajectory. (a) The adjustable finite-element mesh, (b) a computed free-jet profile, and (c) A comparison of finite element solutions and experimental data.

In Figure D-64, graphical representations of the finite element solution for flow over a spillway Ogee crest are provided. It shows color-representation of velocity magnitudes, velocity vectors, and particle trace lines.

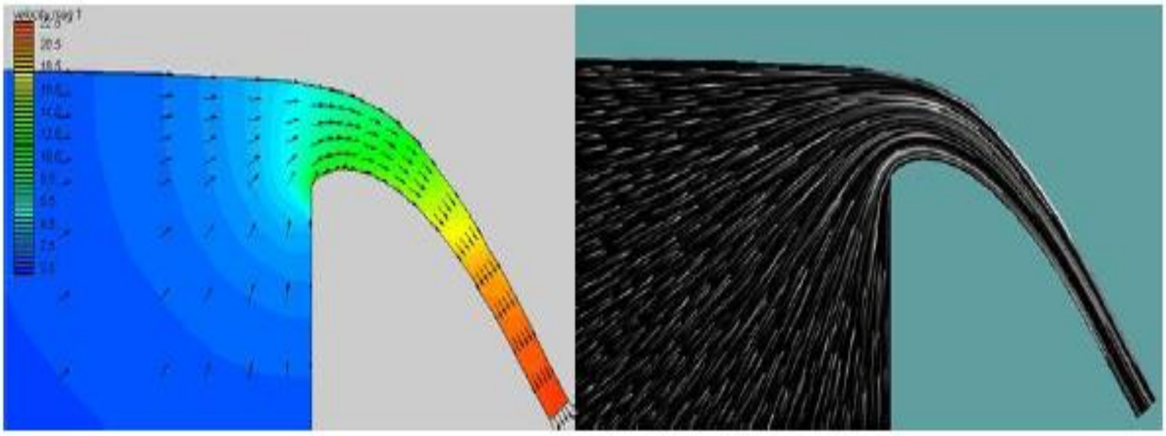


Figure D- 64 Graphical representations of flow over an Ogee crest by finite element simulation.

The finite element method has also been applied to solve seepage hydraulic problems. One example is the application of the method to analyze the unsteady saturation of an earth fill dam during initial reservoir filling [102]. The computational mesh of the finite element model is shown in Figure D-65a. The simulated phreatic lines at various reservoir levels during the filing are shown in Figure D-65b.

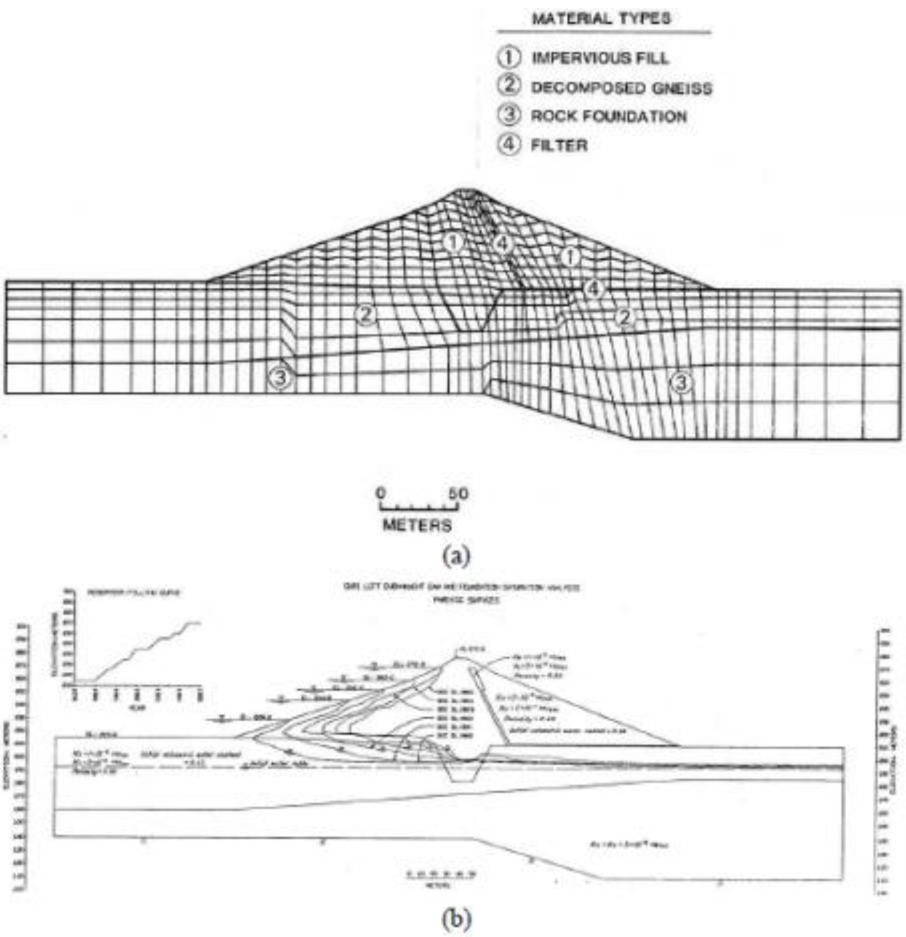


Figure D- 65 Simulating saturation of an earth fill dam during initial filling by finite element method: (a) the finite-element grid; and (b) simulated phreatic surfaces at different time.

Solving flow problems by finite element method with unstructured-grid network.

A hydrodynamic and water quality simulation model system of the Chesapeake Bay was developed using a three-dimensional unstructured grid and available bathymetric data. The model employs a semi-implicit finite-element/finite-volume method. The modeling system has capability to simulate saltwater intrusion and the coastal plume. It uses high-resolution hybrid triangular-quadrangular unstructured-grid with a flexible vertical grid system and an implicit scheme that allows use of larger time steps. The model provides realistic results on all hydrodynamic variables including processes such as the channel-shoal contrast in the estuary and plume propagation in the coast. The horizontal-grid network is shown in Figure D-66. Vertical grid networks for selected locations are shown in Figure D-67. A sample simulated flow patterns (arrows represent cross-channel flow and colors represent along-channel velocity) are shown in Figure D-68a. Some simulated flows at selected transect measured in (a) September and (b) November 1996 are depicted in Figure D-68b (Looking upstream, shaded areas indicate up-estuary flow perpendicular to the transect direction; vectors indicate lateral flow) [71].

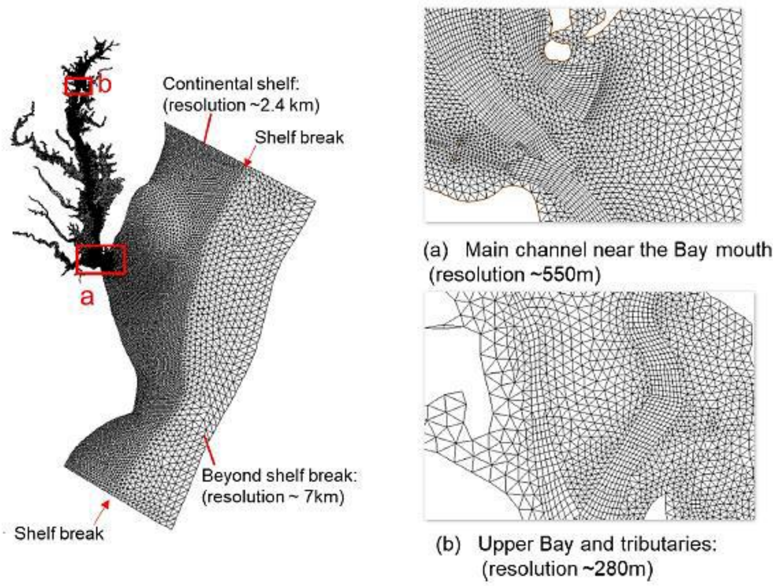


Figure D- 66 The horizontal-grid network for the Chesapeake Bay study.

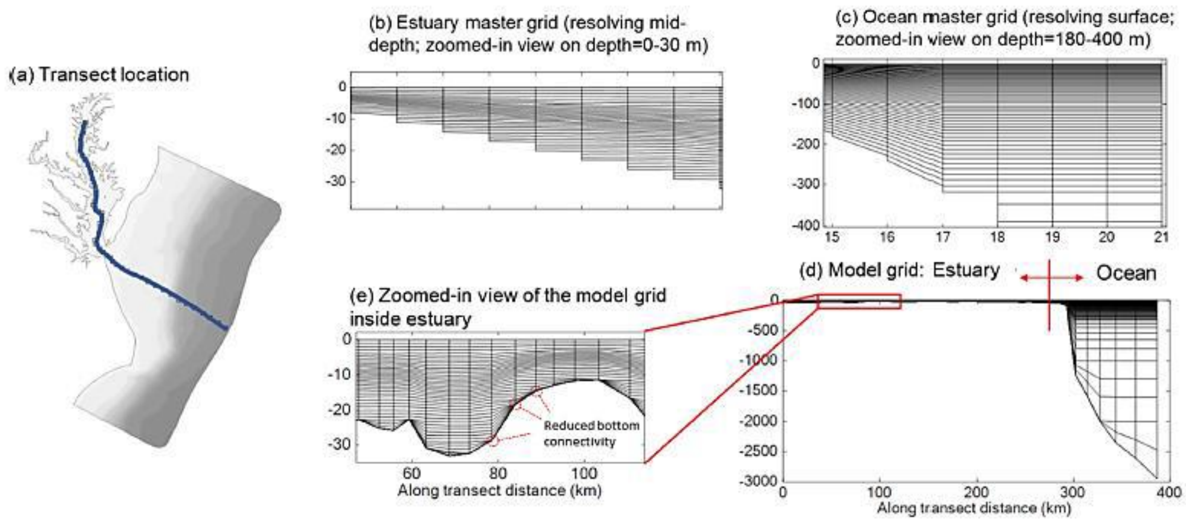
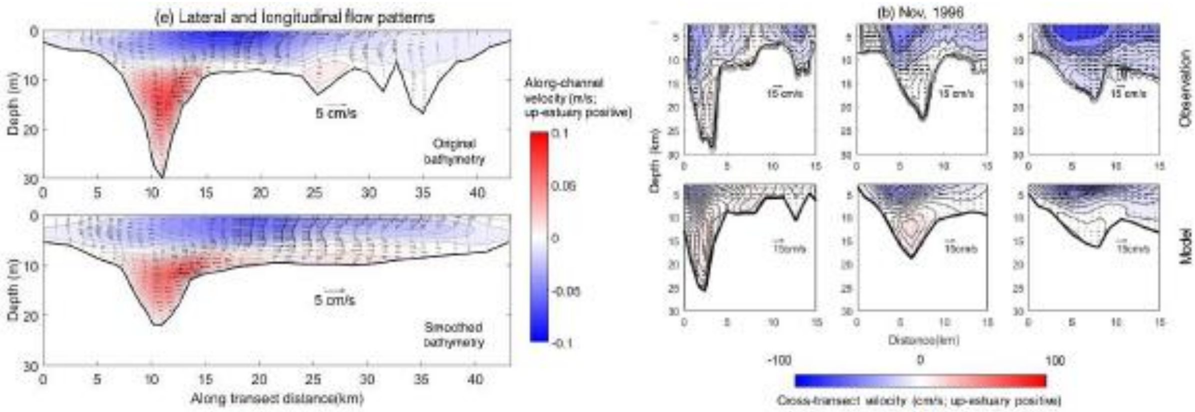


Figure D- 67 Example vertical grid networks for the Chesapeake Bay study.

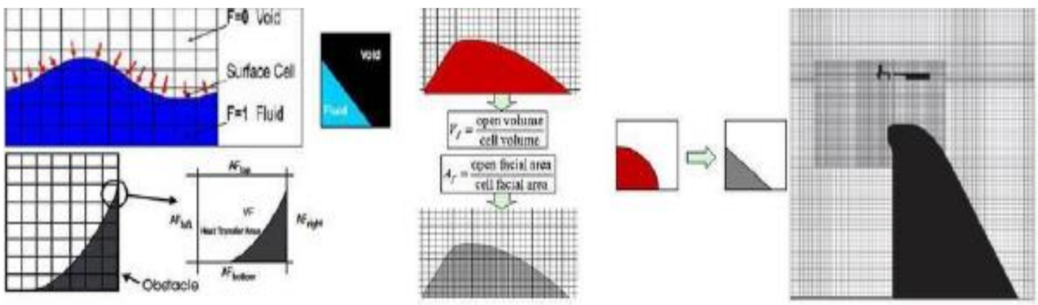


(a) (b)

Figure D- 68 Chesapeake Bay study. (a) Example simulated flows. (b) Comparison of observed and simulated flows.

13. Numerical Modeling based on the Volume-of-Fluid Method

Volume-of-Fluid Method (VOF) is a numerical method of simulating free surface dynamics. It is classified as Euler method. Euler methods are based on a computational grid, which can be either stationary or non-stationary. In case of non-stationary grid, the grid motion is determined by the change of the surface shape [130]. A free surface is an interface between water and air in which the air can only apply a pressure on the water. The VOF method consists of three elements: a scheme to locate the free surface, an algorithm to track the surface as a sharp interface moving through the grid, and a means of applying boundary conditions at the surface. The VOF concept, such as FAVOR of Flow-3D [132], treats interface by fluid fractions. It calculates the area-fraction (AF) and volume-fraction (VF), in each staggered cell and the ratios are integrated into conservation equations as depicted in Figure D-69a. An example geometry and grid of a dam spillway is shown in Figure D-69b [112, 69].



(a) (b)

Figure D- 69(a) Illustrative treatment of interface by fluid fractions and (b) example geometry and grid of a spillway.

The Volume-of-Fluid (VOF) Method has been applied successfully by engineers to analyze many complicated surface-water hydraulic problems in designing or modifying hydraulic structures. One of the most commonly used VOF models is FLOW-3D. The FLOW-3D model is a relatively accurate and robust free-surface simulation CFD software system, and an ideal tool for modeling dam and spillway structures.

Many dam safety professionals and design engineers around the world use FLOW-3D for quantifying all aspects of dam and spillway design, operation, and maintenance. With this model, it is relatively simple to generate rating curves and detailed velocity profiles for complex spillways, including the effects of air entrainment and transport. It also enables the quantification of structural stresses (pressure and shear forces) under normal and extreme operational conditions, including transient effects due to gate operation. FLOW-3D has been widely used to confirm the hydraulic performance of proposed spillway designs and to help professionals meet dam safety requirements. An example of applying VOF method to analyze spillway flip buckets is depicted in Figure D-70.

Simulation of the flood due to a dam break has been an integral part of the dam safety program. The results have been used to prepare Emergency Action Plans (EAP). Traditionally dam break analyses are performed based on 1-D and/or 2-D models. However, in recent years, more dam break analyses are carried out using three-dimensional CFD models especially the VOF based models for their ability to simulate complicated highly irregular free surfaces.

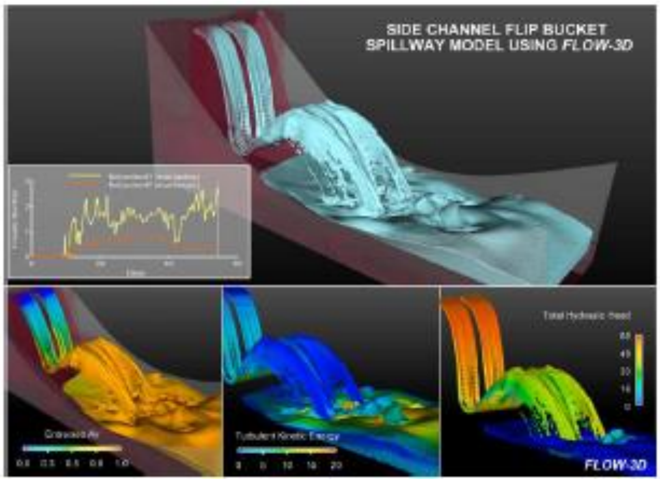


Figure D- 70 Flip bucket simulation using Flow-3D VOF model.

14. CFD Modeling for Evaluation of Hydraulic Structures

As indicated previously, due to the climate change, extreme floods have been occurring more frequently in recent years and the design floods of many dams worldwide have been revised. It requires many spillways and their associated hydraulic structures to be updated to handle the increased flood discharges. Traditionally, reduced scale physical models were used in hydraulic laboratories to study the hydraulic performance of spillways and related structures. However, many scale model studies are prone to problems associated with the scaling effects, and are difficult to capture behaviors such as cavitation and air-entrainment effects. With the advancement in computing and software technology, the performance of hydraulic structures can be investigated numerically with a reasonable level of confidence. As an example, a number of spillway upgrade projects in Australia have been performed using 2-D and 3-D CFD models for spillway upgrade projects [69]. They were conducted to investigate spillway related hydraulic behaviors of hydraulic elements such as: apron or dissipator; flip bucket; plunge pool; hydraulic jump; shock waves; erosion; bridge piers; pressure, velocity and flow depth distributions; overtopping or submerged flow; orifice flow; and re-evaluation of rating curves etc.

Ruby Dam was constructed in the 1930‘s in Madison County, Montana on the Ruby River, a tributary to the Jefferson River, one of the three rivers that form the Missouri River at Three Forks, Montana. The dam is a high-hazard earth-embankment dam with multiple flow control structures including a low-level outlet and a principal spillway. After 75 years of operation, rehabilitation of the structure was performed. A new 6-foot diameter steel-lined outlet conduit was constructed through the downstream discharge tunnel, and extended to a new valve house located at the downstream toe of the dam, where two new jet flow gates were installed to regulate flow. These jet flow gates discharge water at relatively high velocity into a Type III stilling basin. The stilling basin (Figure D-71) was designed and tested based on a CFD model [131].


Figure D- 71Ruby Dam outlet works under construction and operational testing.

A CFD modeling task may include the following steps.

1. Pre-processing to prepare/develop the CFD model
    a. Review all available relevant information such as drawings, topographic data and published physical model test results and prototype observations. Identify likely flow types, special requirements for inclusion in the model.
    b. Import and create topographic data and spillway geometry. Determine model extent and generate suitable computational grid.
    c. Assign boundary and initial conditions. Assign appropriate roughness to non-flow surfaces. Select fluid properties, turbulence model and other physics models.
2. Model validation
    a. Compare computed results with published and other available data.
3. Simulation and analysis
    a. Carry out parametric study, e.g. different combination of headwater and tailwater levels, different gate opening configurations, geometrical changes to the approach condition.
4. Post-processing and interpretation
    a. Extract results such as flow rates, velocities, pressure, water surface profiles, vorticities, and forces on structures.
    b. Interpreted results, e.g. cavitation index, head loss, hydraulic grade lines, stream power, performance evaluation and ranking.

There are still limitations and rooms for improvements in CFD modeling techniques that require further research and development. The areas for further CFD enhancement may include: cavitation; air-entrainment effect; air demand; thin jet and breakup of jet; fluctuating pressures at spillway and apron floor; dynamic interaction; scour modeling; and overland flow etc. [69].

15. Examples of CFD Modeling for Analyses of Hydraulic Structures

CFD modeling technology has been advancing rapidly over the past years. Many advanced CFD software packages have been developed and continuously improved in terms of capabilities in duplicating the real-world hydraulics and hydrodynamics. Many of them, such as FLOW-3D, have been widely used to analyze dam and river problems with relatively high degree of success. They have been applied to perform [132]:

- Spillway flow analysis
  - Simulating of free discharge and submerged ogee spillways.
  - Simulating of standard and non-standard Parshall flumes.
  - Simulating of partially and fully-submerged gated openings.
  - Simulations of labyrinth spillways.
  - Simulating of stepped spillways.
  - Simulating of tunnels, pipes, turbines, and draft tubes, etc.
- Nappe impingement analysis.
- Pressure/shear distribution and tractive force analysis.
- Tailrace, stilling basin and energy dissipator analysis.
- River reach, approach flow, and intake analysis.

Recent developments have allowed simulations of sediment transport and deposition with satisfactory results. CFD capabilities to model cavitation formation and damage for dams and tunnels and air entrainments have also been developed [132]. Multiple tools are required to create a computational mesh. A bathymetric surface of the river must be created from river point surveys and contoured data, and a CAD developed for the engineered structures. These data are then used to create the computational mesh. The resulting computational mesh is then used to run a CFD flow solver [55]. Hydraulic design of turbines can be developed by means of CFD simulations and experiments. Building on experience and know-how from extensive model and prototype testing a large proportion of turbines has been designed and optimized by use of CFD only for more than two decades. [133].

Various CFD modeling packages have been applied to perform hydraulic analysis of dam projects. For instance, the GHD consulting firm [122] which was established in 1928 has used CFD packages such as FLUENT and ANSYS CFX to perform hydraulic modeling and analysis for the design and assessment of dam projects. The CFD modeling studies are conducted to: assess flow behavior with respect to spillway arrangements; supplement design optimization; develop static and dynamic loading as an input to structural design; provide assessment of flow conditions in and around fishways to optimize their effectiveness; and assess erosion potential around hydraulic structures. Some exhibits of the CFD analysis by GHD are shown in Figure D-72.

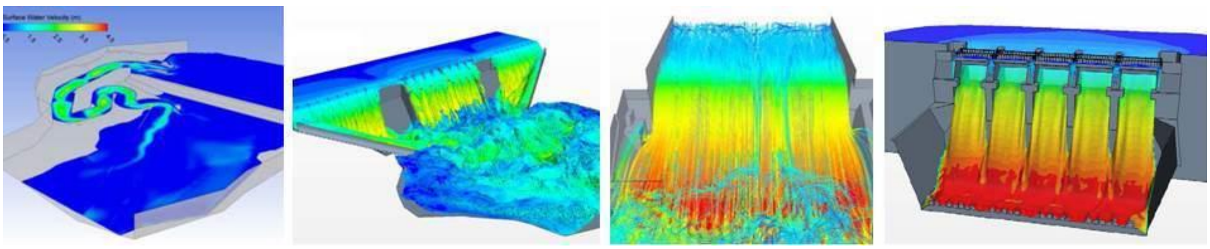


Figure D- 72 Sample exhibits of GHD CFD modeling results.

The Dalles Dam spillway CFD analysis

Computational fluid dynamics (CFD) models of the spillways at the John Day and Dalles Dams were developed to investigate the velocity field near the tainter gate. These models use the Flow-3D commercial program to solve the Reynolds-averaged Navier-Stokes equation to simulate transient free-surface flows. To reduce the computational effort required to operate the CFD model, the model domains were reduced to a single 2-D plane that passes through the centerline of the bay. This approximation is appropriate for understanding hydraulic phenomenon in close proximity to the tainter gate centerline (Figure D-73), however differences between the CFD model and prototype would be expected near the piers on either side the tainter gate. Several 3-D phenomena occur near these piers, including sometimes large vortices, which will not be captured by this 2-D model. In addition, far upstream of the gate, the lateral flow component has been shown to be significant depending upon powerhouse and spillway conditions. Therefore, if results far upstream of the gate are required, a 3-D model of the fore bay that incorporates these lateral flows should be utilized [59, 60].

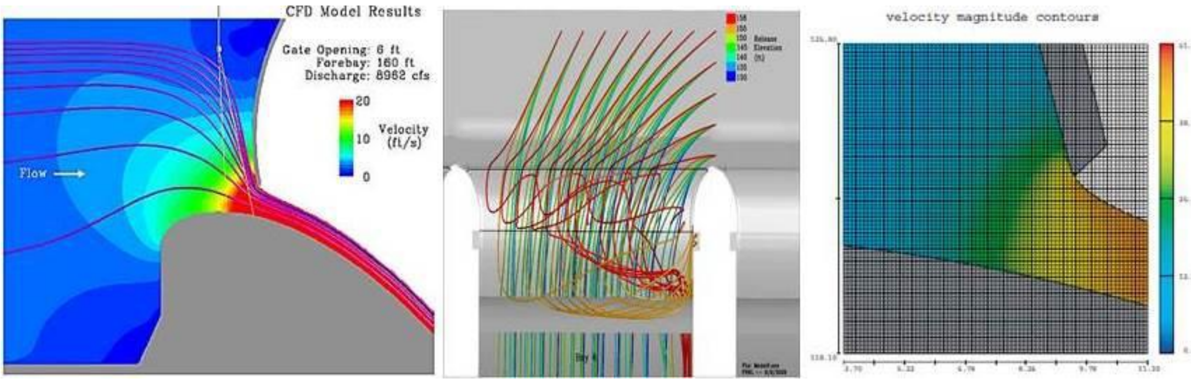


Figure D- 73 Simulated velocities, streamlines and CFD mesh for a spillway gate at the Dalles Dam

Laleli dam spillway CFD numerical model investigation

Laleli dam is a 127.5-m high roller compacted concrete (RCC) dam located on the upper reach of the Coruh River in the northeastern region of Turkey. The 38-m wide spillway is equipped with four radial control gates and a flip bucket located at the downstream end. Due to the steep slope of the spillway and high velocities, cavitation risk was one of the main concerns. The hydraulic characteristics of the spillway were investigated using a 3-D CFD simulation model supplemented by physical model tests [112]. The CFD model was developed using the commercially available Flow-3D software package which is based on the Volume-of-Fluid (VOF) technique. The model solves the Reynolds averaged Navier-Stokes Equations coupled with a turbulence model. Flow characteristics such as flow rate, flow depths, water surface profiles, pressure distributions, vertical velocity distributions, air entrainments, scale effect and cavitation potential of the spillway for both model and prototype scales were investigated. A 1:25 scale physical model (Figure D-74) was constructed and tested to provide additional data to supplement the CFD study. This physical model was designed and built for the 1,000-year flood condition with a maximum discharge of 1,023 m^3/sec. The validation of the CFD model was made using the physical model test data from the model-scale tests. The scale effects were investigated by making additional prototype-scale CFD simulations and compared the results with that of the model-scale simulations. The air entrainment model of the Flow-3D was used to analyze the spillway cavitation potential and air entrainments induced by the aeration device of the spillway. As one can expect, the simulation results indicated the air entrainments were stronger for the prototype-scale condition compared with the model-scale condition. The CFD simulation results indicated that the effect of air entrainment on the pressure distribution on the spillway is negligible. In addition, the CFD simulations made under both prototype- and model-scale conditions showed similar pressure distributions. The parameters for designing an aeration ramp are shown in Figure D-75a. The CFD computational grid and a schematic of the initial condition of the model are shown in Figures D- 75c and D-75d respectively. The results were used to finalize the design of the ramp which was tested in the physical model as shown in Figure D-75b.

Figure D- 74 The Laleli Dam Spillway 1:25 physical model (left) and measuring points (right).

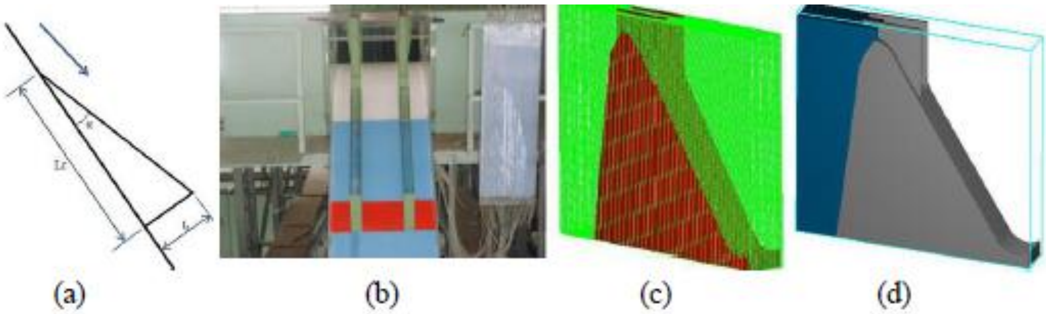


Figure D- 75 (a) aeration ramp design parameters; (b) the model of the final design; a grid of the CFD model; and (d) a schematics of the initial condition for CFD simulation.

An example simulated volume fractions of entrained air in model and prototype-scales by the CFD model are shown in Figure D-76.

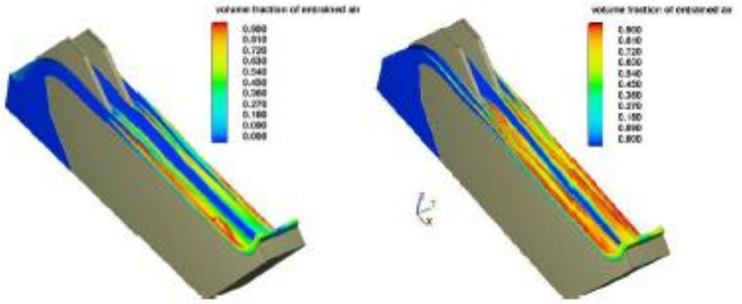


Figure D- 76 Volume fractions of entrained air in model-scale (left) and in prototype-scale (right).

Limestone and Wuskwatim Generating Stations Model Studies

Based on the data obtained from physical spillway model studies for several generating stations operated by Manitoba Hydro, a comparison study of discharge rating curves obtained through numerical modeling with CFD software Flow-3D was performed. These spillways have significantly different hydraulic characteristics in terms of height and design head and are well suited for this comparison study. The Flow-3D model results were found in general agreement with physical model data. Comparisons of discharge rating curves from both types of models for Limestone and Wuskwatim Generating Stations are briefly described as follows [66].

The 1,340-MW Limestone generating station is located in northern Manitoba on the Nelson River and was completed in 1990. The spillway consists of 7 bays, 13-m wide each. A physical model study conducted by Western Canada Hydraulics Laboratories provided the test data. The model spillway included a flip-bucket for dissipation of energy. This geometry differs from the CFD model that is based on the actual constructed spillway which utilizes a stilling basin for energy dissipation. This difference was considered insignificant regarding the discharge comparison as the flow is supercritical well before reaching any variations in geometry. The model study results are shown in Figure D-77.

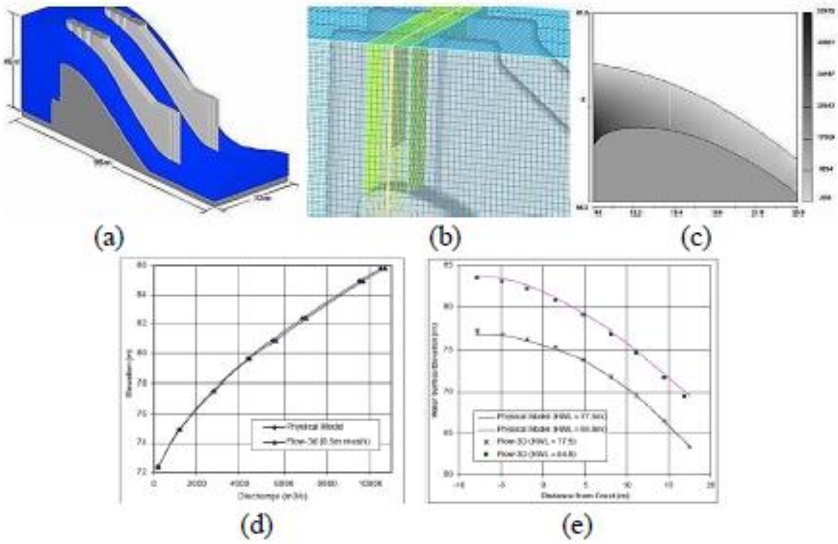


Figure D- 77(a) Spillway modeled by Flow-3D. (b) The CFD mesh for the gated crest. (c) Simulated pressure distributions by Flow-3D. (d) Spillway discharge rating curves from the physical and Flow-3D models. (e) Observed and simulated water surface profiles.

The 206-MW Wuskwatim generating station is located on the Burntwood River in northern Manitoba and the construction began in 2006. In the design of overflow spillway, information regarding the hydraulics of the flow over and around the structure was of interest. The hydraulic data used in the study includes discharge rating curves, pressures over the rollways and on the piers, water surface profiles, and velocity profiles. A 1:36 scale physical model study of the preliminary design of the spillway was conducted at the University of Manitoba. One of the concerns during the study was that as fore bay levels exceed design water levels, pressures over the spillway crest can become negative. If these pressures get too low, cavitation may occur and cause significant damage to the spillway surface. Knowledge of water surface profiles is important for determination of appropriate pier heights such that overtopping does not occur. Based on the test results, a flip bucket for energy dissipation was not incorporated in the final design. Information based on the fully open single bay overflow spillway was used in the CFD comparison. The model study results are illustrated in Figure D-78.

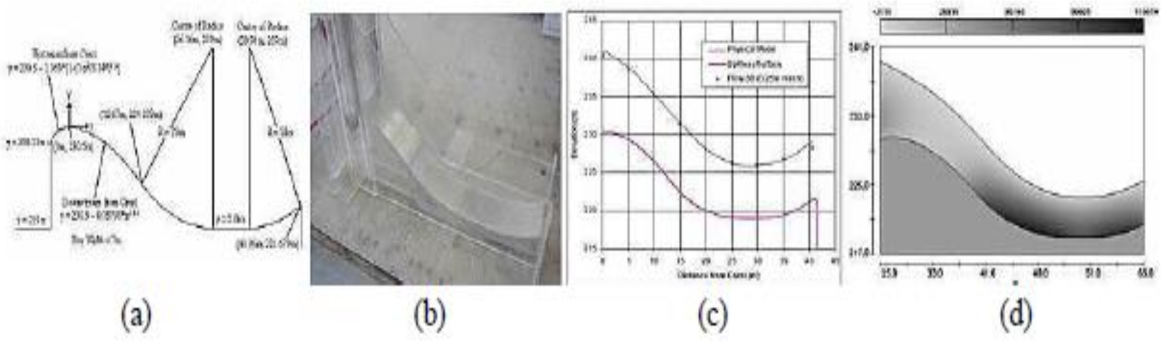


Figure D- 78(a).The overflow spillway with flip bucket. (b) The physical model. (c) Observed and simulated water surface profiles. (d) Simulated pressure distribution by Flow-3D.

Simulation of Pressure Surge in an Elbow Draft Tube

One of the most difficult problems in the operation of hydro power plants (especially with Francis turbines) is the appearance of a vortex rope in the elbow draft tube under part load conditions. The rotation of the vortex rope causes severe pressure fluctuations. In addition to these pressure fluctuations in elbow draft tubes under certain conditions, a synchronous component of the pressure surge can cause operational problems. This synchronous fluctuation acts as an excitation which can introduce discharge and pressure oscillations in the water system. It can lead to intolerable pressure surges and forcing changes of operation rules. These oscillations are difficult to assess by physical model tests [118-120]. A three-dimensional CFD simulation model was developed to investigate the hydrodynamic characteristics of an elbow draft tube under part load conditions. The CFD model was initially tested on a straight diffuser with satisfactory results. The turbulence model enabled the investigators to assess the formation of the vortex rope. The calculated pressures were in good agreements with the measurements and the calculated frequency of the vortex rope agreed extremely well with the measured ones. The amplitudes of the pressure fluctuations near the draft tube inlet also agree quite well with the measurements. The water hammer caused by the synchronous pressure surge was analyzed by applying a one-dimensional model based on the method of characteristics. The CFD model was developed based on the computer code FENFLOSS, developed at University of Stuttgart. FENFLOSS is a finite element flow simulation program based on the Reynolds averaged Navier-Stokes equations with various models of turbulence. FENFLOSS uses a segregated solution algorithm, in which the individual momentum equations are solved separately. The time discretization is done with a fully implicit scheme of second order.

The subject elbow draft tube consists of three outflow channels with two dividing piers. At the inlet the computational domain is enlarged up to the runner hub. The computational grid (Figure D-79a) consists of about 200,000 nodes. The results show a strong transient behavior in the draft tube. Figure D-79b presents a cork-screw like vortex rope as an iso-surface of the pressure. The changing size of the vortex rope indicates strong pressure surges. An example of simulated and observed pressures near the inlet of the draft tube is shown in Figure D-79c for comparison.

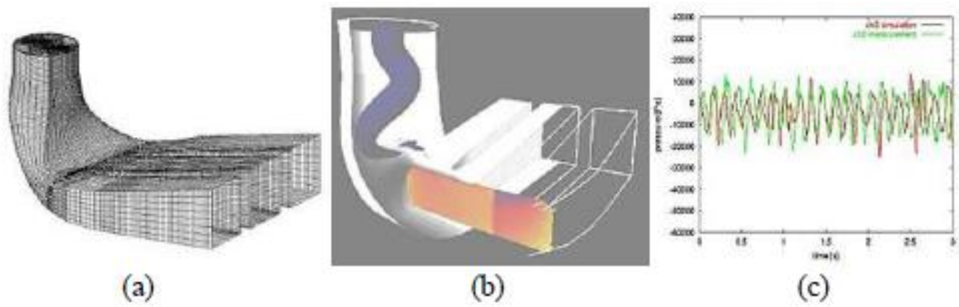


Figure D- 79(a) Grid of the draft tube; (b) simulated vortex rope in the draft tube; and (c) a comparison of simulated (red) and observed (green) pressures near inlet of the draft tube.

Numerical Flow Simulation of Pool-Type Fishways

Fishways are built to restore the connectivity of rivers and allow the migration of aquatic fauna. In order to assess the performance of a pool-type fishway, it is necessary to understand its flow structure, since general observations of fishways can only provide a rough idea of the actual conditions inside the pools. The traditional laboratory studies using flow measurements and visualization techniques are often considered insufficient for the design. In recent years numerical simulations are often used to supplement the design of fish passage structures, such as pool-slot and vertical-slot type fishways, to improve its hydraulic performance. CFD modeling using fixed computational grids has encountered difficulties in modeling the overfill geometry as well as the number of pools and the discharge. Fishway hydraulics is often characterized by an irregular free water surface that requires special treatments in the numerical simulations. The FENFLOSS CFD model as developed by the University of Stuttgart uses fixed computational grids and is almost exclusively applied for modeling of closed systems like water turbines or draft tubes [67]. In this study, fishways with a cascade of several pools was investigated to determine the effectiveness of the inlet and outlet designs with respect to the flow conditions in the pools. Four pools were included in the modeling of the fishways. CFD models using CFX software package were also developed and tested and the results were compared with that of FENFLOSS models. Based on this comparison, the CFX software was adopted for the subsequent investigations due to its ability to simulate the irregular free surface geometry. The simulated free surfaces by the CFX models are very similar to the observed conditions. An entire vertical slot fishway was simulated to compare the advantages and disadvantages of different types of designs. It modeled the fishway from inlet to outlet and included a number of baffles and bends. In general, guide elements are installed in pool-type fishways to provide smooth flow through the slot. The energy dissipation causes a decrease of the maximum velocity in the slot, thus easing the upstream migration for fish.

Three different arrangements of the guide elements in a vertical slot fishway were tested. As shown in Figures D-80 simulation of cascading flow through a vertical slot fishway and flow patterns for three different slot arrangements are depicted.

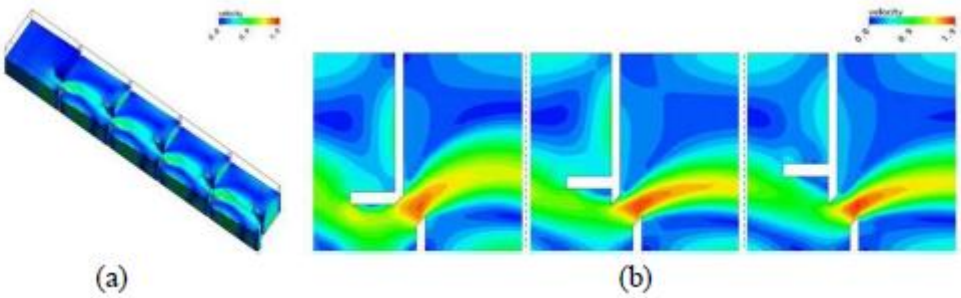


Figure D- 80(a) simulated cascading flow through a vertical slot fishway, (b) simulated flow patterns in a vertical slot fishway for three different slot arrangements.

As shown in Figure D-81, the fishway investigated consists of two bends located at the upstream and downstream ends of the center straight section. Two design alternatives of the vertical slots were investigated with the first (upstream most) slots located on the inner or the outer wall of the fishway. As shown in the figure, the simulated velocity distribution around the slot can be examined in detail using a post processing visualization software.

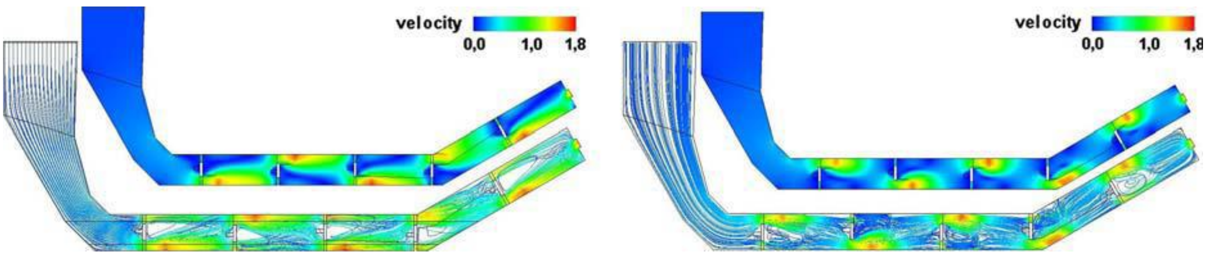


Figure D- 81 Fishway with first slot on the inner wall (left) versus first slot on the outer wall. (Right).

As shown in Figure D-72a, the highest velocities are found directly behind the slot and the velocity is not uniformly distributed over the slot height and decreases toward the surface. In Figure D-82b, a comparison of slot flow-lines for designs with (right) and without (left) guide element is provided.

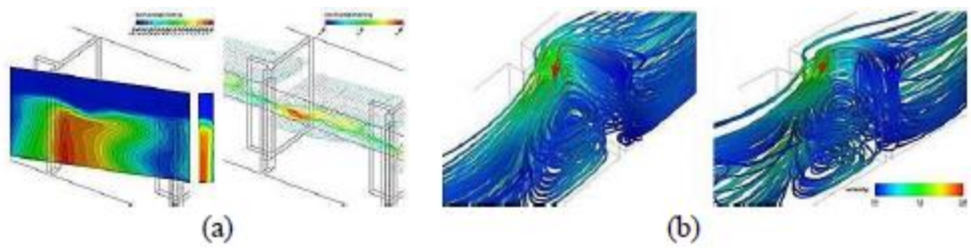


Figure D- 82(a) Velocity distribution in the slot: in direction of the main flow (left); at the slot exit (middle); at the water surface (right), (b) streamlines at the slot: without guide element (left) and with guide element (right).

The computed velocity vectors showing the impacts of the guide position on the orientation of the main flow entering the pool are shown in Figure D-83.

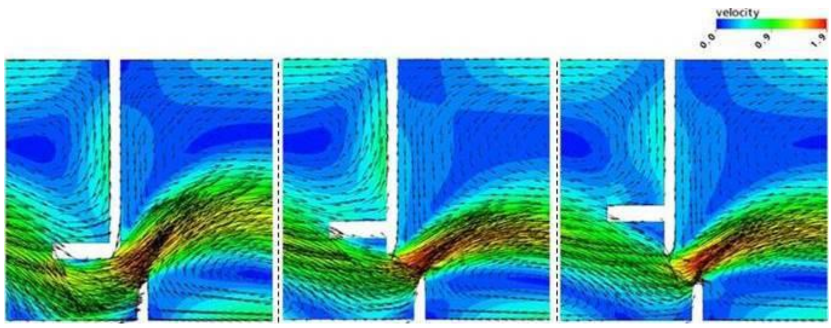


Figure D- 83 Velocity vectors showing simulated flow patterns at different guide element positions.

16. Hydraulic Modeling at Some Hydraulic Laboratories

USBR Hydraulics Laboratory - Refer Figure D-84.

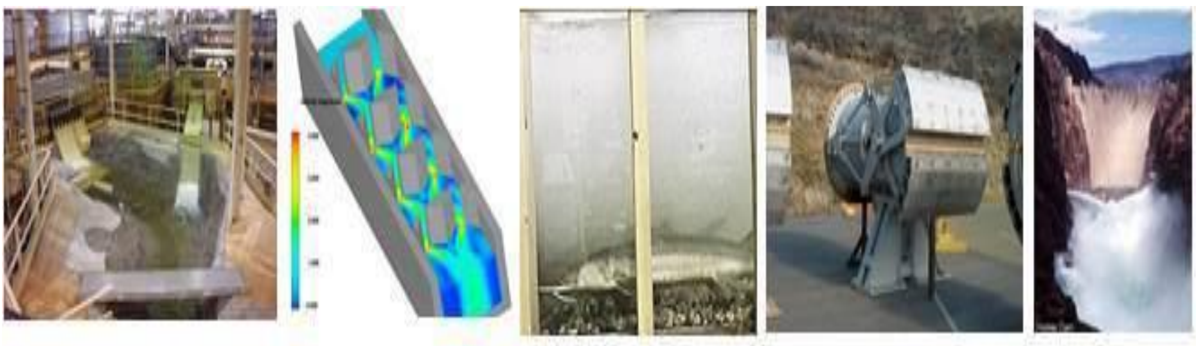


Figure D- 84 USBR Hydraulics Laboratory.

Established in 1902, the Bureau of Reclamation (USBR) is best known for the dams, power plants, and canals it constructed in the 17 western states. USBR, which is the second largest producer of hydroelectric power in the United States, has constructed more than 600 dams and reservoirs including Hoover Dam on the Colorado River and Grand Coulee on the Columbia River. USBR's work is focused on ensuring the safety of dams, managing and conserving water resources, constructing, operating and maintaining essential civil and hydromechanical infrastructure, and protecting and improving the environment. The USBR Hydraulics Laboratory is specialized in hydraulic modeling, analysis, and laboratory and field measurements and provides solution of water resources, hydraulics, and fluid mechanics problems. The laboratory has nearly 90 years of experience in developing state-of-the-art hydraulic engineering technologies. The laboratory performs work for USBR and other organizations, including international clients, federal, state, and local governments, and private entities. Based on the work of the laboratory, USBR has published many widely referenced manuals, guidelines, books, monographs, and reports in hydraulic engineering. Example publications include "Design of Small Dams,""Hydraulic Design of Stilling Basins & Energy Dissipators,""Cavitation in Chutes and Spillways,""Air-Water Flow in Hydraulic Structures,""Guidelines for Hydraulic Design of Stepped Spillways," and "Water Measurement Manual" etc.

The Hydraulics Laboratory conducts hydraulic investigations in the following areas.

- Hydraulic Structures Modeling
  - Physical hydraulic model studies are conducted in a 54,000 sq. ft. indoor lab facility.
  - Flow capacities up to 60 cfs and 600 feet of pressure head are available.
  - Uses fixed and variable-slope flumes, low-ambient pressure chamber, and the latest data acquisition equipment.
- Computational Fluid Dynamics Modeling
  - CFD software such as FLOW-3D is applied to complement physical modeling capabilities.
  - Initial investigations with CFD are generally used to narrow the range of alternatives tested in a physical model.
  - Flow properties that are difficult to physically measure can be evaluated numerically.
- Water Conservation, Water Management, Water Measurement
  - Canal structures, intakes, and diversions.
  - Canal operations, automation, and modernization.
  - Water measurement structures, design and calibration.
  - Assist irrigation districts with remote monitoring and control of irrigation systems.
  - Provide training to canal operators using indoor model canal simulator.
  - Flow measurement training, including the use of software tools for design and calibration of flumes, weirs, and gates, etc.
- Environmental Hydraulics
  - Fish protection, screening and fish passage.
  - Regulating temperature and water quality of reservoir releases by selective withdrawal.
  - Provide design support and field evaluation of multi-level intake structures.
  - River stabilization and restoration for habitat protection and improvement.
  - Reservoir density current measurements and acoustic imaging.
  - Reservoir and river sedimentation.
  - Total dissolved gas abatement.
- Hydraulic Structures and Equipment
  - Operation and maintenance issues related to hydraulic facilities.
  - Cavitation in spillways and hydraulic machinery.
  - Performance of stilling basins, pumps and turbines, gates and valves.
  - Flow induced vibration and stresses.
  - Hydro turbine hydraulics.

- Dam and Infrastructure Safety
  - Dam Safety improvements.
  - High capacity spillways such as labyrinth weirs and fuse plugs.
  - Dam overtopping protection.
  - Dam and embankment breach modeling, canal breach modeling.
  - Soil erodibility measurements and research.
  - Spillway erosion modeling.
- Field Measurements and Testing
  - Discharge measurements using Doppler and transit-time acoustic instruments.
  - Emergency gate closure testing.
  - Scanning sonar underwater imaging.
  - Channel bathymetry.
  - Flow velocity in 1, 2, or 3 dimensions, using ADV's and ADCP's.
  - Pressure, (dynamic and steady-state).
  - Stress, strain and vibration in mechanical components of hydraulic machinery.
  - Temperature and water quality in reservoirs and rivers.
  - Air demand of hydraulic equipment.

Coastal Hydraulics Laboratory - Waterways Experiment Station (CHL-WES) - Refer Figure D-85.

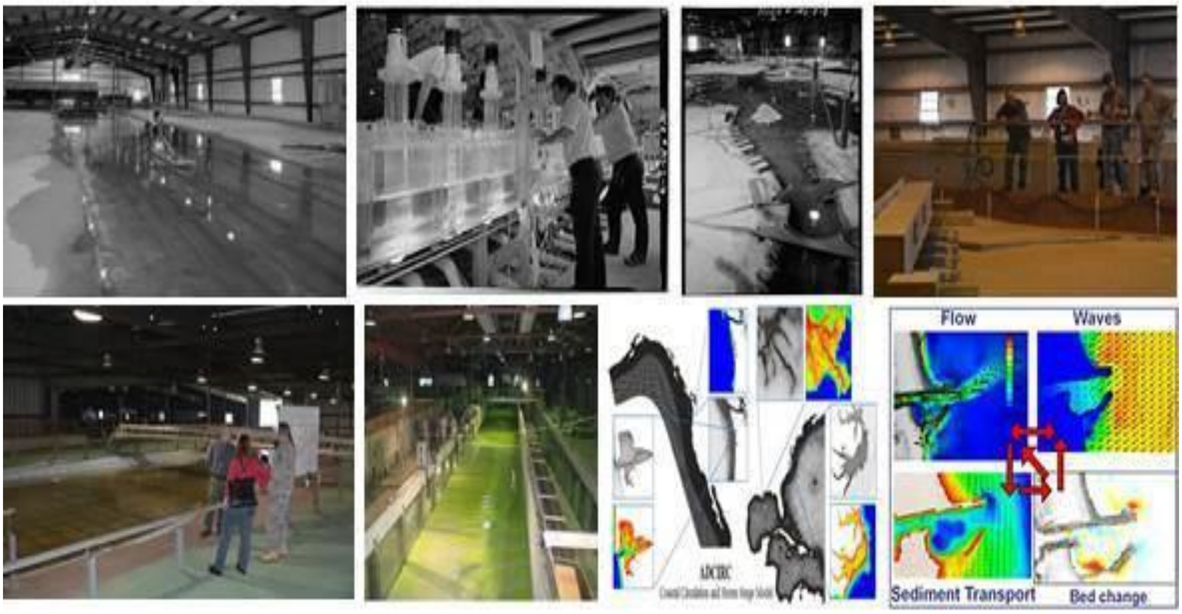


Figure D- 85 Coastal & Hydraulics Laboratory - Waterways Experiment Station (CHL-WES)

The Waterways Experiment Station (WES) in Vicksburg, Mississippi was established through the US Flood Control Act of 1928. The Hydraulics Laboratory (HL) of WES is the first federal facility to apply modern research methods to large, nationally significant Civil Works projects. The laboratory was envisioned to serve multiple agencies as well as the public and has been developing modeling tools for supporting a broad spectrum of water resources concerns. In 1932, HL expanded into coastal physical modeling and the Coastal Engineering Research Center (CERC) was created in 1963 to focus on coastal engineering research. In 1996, the Coastal and Hydraulics Laboratory (CHL) was created through the merger of CERC and HL. In 1999, with formation of the US Army Engineer Research and Development Center (ERDC), which encompassed WES, CHL became one of the seven ERDC laboratories. Today, CHL leads coastal, estuarine, and hydraulic water resources research in both Civil Works and military domains.

CHL has been continuously exploiting advanced computational sciences and emerging computing systems for coastal and hydraulics applications, and leveraging foundational research in artificial intelligence and machine learning for solving problems in the coastal and hydraulics domain. Specifically, applying research and new technologies in the areas of advanced computing, big-data analytics, remote sensing, data assimilation, and autonomous systems applied to water problems.

For issues of water security, climate change, navigation, and watershed analysis, CHL tools integrate across scales (large and small, space and time scales) and disciplines, coupling environmental, geotechnical, arctic, and socio-cultural processes. CHL has developed decision support tools that provide uncertainty/risk analysis and studies of resilience in a changing climate.

CHL research and development addresses water resource challenges in groundwater, watersheds, rivers, reservoirs, lakes, estuaries, harbors, coastal inlets and wetlands. Their concerns include: hydraulic structure and waterway design/operation, sediment dynamics, multiphase flow, nonlinear waves, turbulence, and energy dissipation, effects of navigation traffic on waterways, water and environmental quality management, contaminated groundwater resources, wetland maintenance and management, flood-control channel design and operation, control of overland, bank and near-structure erosion, and watershed runoff and flow analysis. These developments include creation and/or enhancement of computational and process understanding within the models, and improved modeling productivity through the coupling of these models with comprehensive and consistent graphic user interface, visualization, and parameter estimation methods. Linkages to Geographic System (GIS), optimization and uncertainty methods, and decision support techniques are also key components of these system developments. The developed modeling systems can increase the efficiency and efficacy of modeling to both modeling specialists and decision-makers/managers.

Hydrology

In conjunction with ERDC's Environmental Laboratory, CHL provides hydro-environmental modeling, tools and analysis for environmental compliance, and other water quality analysis. CHL analyzes and simulates surface and subsurface hydrologic systems and coupled groundwater and surface water systems. CHL hydrologic tools are integrated with other simulation capabilities, such as weather and storm surge forecast models and systems.

River and Estuarine Engineering

As the first federal hydraulics research facility, CHL has developed significant core competencies include the development and application of advanced instrumentation and sensing equipment, field data collection techniques, numerical models, and physical modeling to solve problems related to river and estuarine hydraulics, sediment transport, geomorphology, salinity intrusion, and marsh evolution.

Coastal Engineering

To support mitigating coastal-zone hazards such as storms, which result in extreme coastal erosion, flooding, and infrastructure damage, CHL has advanced a range of measurement and modeling expertise, tools, and databases to characterize the relevant coastal forcing climatology. To quantify the coastal response, CHL develops advanced field measurement techniques and modeling/analysis tools for coastal sediment transport and morphology response to support integrated coastal zone management and resilient coastlines. To support the design of engineering measures to manage coastal risk related to navigation, flooding, and coastal erosion, CHL develops and executes physical and numerical modeling of coastal protection structure response.

Fluid-Structure Interaction (FSI)

CHL develops and maintains advanced computational modeling tools for FSI and conducts research in a range of areas that support continued advances in FSI modeling: numerical analysis, scientific computing, engineering mechanics, and mathematical modeling. CHL also develops and maintains an array of experimental facilities (flumes and basins) and field data collection capabilities, which are used to study processes, validate computational models, and design structures and vessels. In support of FSI simulation, CHL also conducts specialized research and development into experimental methods, data acquisition, and field data collection to advance its capability to measure more complex processes and achieve higher accuracy with new measurement techniques. In support of maritime operations, CHL delivers analytic tools and predictive capabilities. CHL combines advanced data analytics derived from integration of large datasets from waterborne cargo records, hydrographic surveys, marine vessel position reports, and dredging vessel diagnostics to determine critical needs.

In the areas of the surface water developments related to estuarine and riverine hydrodynamics, WES provides modeling support of flood-control project operation and salinity and sediment management associated with navigation. The modeling supports include utilization of the WES-developed TABS hydrodynamic/sediment transport modeling system. Two other hydrodynamic models, namely RMA10-WES and CH3D-WES were also developed and applied to study sophisticated three-dimensional hydrodynamic and water quality problems. The RMA10-WES code is a Galerkin-based finite element program that simulates three-dimensional unsteady flows in rivers and estuaries. CH3D-WES (Curvilinear Hydrodynamics in Three Dimensions) is a time-varying, 3-D numerical hydrodynamic model that can be applied to rivers, reservoirs, and estuaries. Enhancement of the graphical user environments for CH3D-WES and RMA10-WES is accomplished through their implementation into the WES Surface Water Modeling System (SMS). The SMS is a graphical user support system for several multi-dimensional surface water hydrodynamic codes. For near-field flow problems, pressure distributions are not hydrostatic due to curvatures of flow. WES has developed a suite of near-field hydrodynamic models which, in concert with physical models, provide a more optimal approach to modeling. The CH3D-WES hydrodynamic model has seen application to Chesapeake Bay on multiple grids. CH3D was substantially modified at the WES and is referred to as CH3D-WES. The model provides computations of surface elevation, velocity in three dimensions, vertical diffusivity, salinity, and temperature [75].

Through CHL, WES has published numerous design guides and reports used by hydraulic engineering professionals worldwide such as: "Hydraulic Design Criteria,"—"Hydraulic Design of Spillways,""Gravity Dam Design,""Arch Dam Design,""Earthquake Design and Evaluation of Concrete Hydraulic Structures,""River Hydraulics,""Hydraulic Design of Reservoir Outlet Works,""Hydraulic Design of Lock Culvert Valves,""Hydrologic Engineering Requirements for Reservoirs,""Reservoir Water Quality Analysis," etc.

Central Water and Power Research Station (CWPRS), Pune, India- Refer Figure D-86.

Figure D- 86 Central Water and Power Research Station (CWPRS), Pane, India.

The Central Water and Power Research Station (CWPRS) was originally established in 1916 by the then Bombay Presidency as a "Special Irrigation Cell" and was taken over by the Government of India in 1936 to take charge of systematic study of various phases of water projects. CWPRS provides hydraulically sound and economically viable solutions to various problems associated with projects on water resources, energy and water-borne transport, and coastal and harbor engineering. CWPRS provides specialized services through physical and mathematical model studies in river training and flood control, hydraulic structures, harbors, coastal protection, foundation engineering, construction materials, pumps and turbines, ship hydrodynamics, hydraulic design of bridges, environmental studies, earth sciences, and cooling water intakes. The research station has provided services to a number of projects in the neighborhood countries as well as countries in Middle East and Africa.

The Reservoir and Appurtenant Structures Group (RAS) of CWPRS undertakes studies for hydraulic design of spillways and energy dissipators, water conductor systems and other appurtenant structures such as protection works, high head gates, sluices and outlets, surge tanks, tunnels, penstocks, galleries, sedimentation and flushing of reservoirs, sediment control and exclusion devices. These studies are carried out with the help of hydraulic models, mathematical models and desk studies. There are four technical divisions under RAS: (1) Spillways and Energy Dissipators (SED); (2) Control Structures and Water Conductor Systems (CSWCS); (3) Sediment Management (SM); and (4) Hydraulic Analysis and Prototype Testing (HAPT).

(1) Spillways and Energy Dissipators (SED). SED division conducts studies for the spillways of dams, irrigation projects, power generation and flood control. Major activities of the division include:

- Approach flow conditions and protection work of spillway;
- Assessment of discharging capacity of control gates;
- Analysis of spillway cavitation potential and design of aeration devices;

- Analysis of training and divide walls, and trunnion beam;
- Assessment of energy dissipating arrangement;
- Scour downstream of energy dissipator and plunge pool design;
- Performance of spillway during construction;
- Assessment of prototype performance;
- Assessment of power intake flow conditions;
- Transient analysis for assessment and design of surge shafts;
- Flow conditions in head race channel / tunnel;
- Flow conditions in tailrace channel / tunnel including draft tube submergence;
- Analysis of uplift and downpull forces, and air demand of gates; and
- Analysis of air water column separation and requirement of air-vents.

(2) Control Structures and Water Conductor Systems (CSWCS). To minimize damage of the turbine runner blades/buckets due to abrasion, CWPRS has conducted studies for various types of desilting basins and several types of desilting basins have been developed.

(3) Sediment Management (SM). This division provides solutions to the problems of sediments associated with the design of head works on the mountain streams in Himalayan and North East regions. The SM division is responsible for developing the run-of-the-river schemes with reservoir designed to disposal of the deposited sediment at regular intervals. Storage capacity of the reservoirs is maintained by proper sediment management such as applying the technique of hydraulic flushing/ sluicing of reservoirs. Model studies are carried out for optimizing the design of various components of hydropower projects including development of run-of-the-river schemes. Mathematical and physical model studies are applied to design sediment flushing devices, energy dissipators, and desilting basin.

(4) Hydraulic Analysis and Prototype Testing (HAPT). HAPT has carried out hydraulic model studies for many design aspects of barrages such as: energy dissipation arrangements in both spillway and under sluice bays; design of the stilling basins; silt excluders; gate operation; and model studies for flood protection works etc.

HAPT has performed many 1-D/2-D numerical model studies and physical model studies for hydroelectric, and barrage projects. HAPT has developed an integrated approach of using 1-D numerical models for long-term simulation of sedimentation in the entire reservoir and 2-D/3-D numerical models for prediction of the near-field conditions such as for reservoir bed profiles around power intakes. To predict the long-term deposition pattern in the reservoirs of run-of-the-river hydropower projects 1-D numerical models are applied and 2-D/3-D numerical models for predicting the sediment deposition levels in the vicinity of hydraulic structures. CWPRS also provides technical supports for river and canal projects including: flow measurements; SCADA assisted automation of gate operations using acoustic flow meters and water level sensors; ADCP flow meters and electromagnetic flow meters with modern telemetry. The River Hydraulics Division undertakes studies related to river hydraulics. The areas of specialization of the division include: river training; bridges; barrages and weirs; river morphology; inland navigation; flood routing; river behavior; intake structures for water supply schemes; stream gauging and sediment sampling; siltation in reservoirs; sediment control and exclusion devices; and mathematical model studies. The mathematical modeling includes: flood routing studies; cubature computations; hydraulic transients in water conducting system; system analysis of water resources; morphological model studies; river regime studies; and stratified flows and hot water recirculation.

Utah Water Research Laboratory (UWRL) - Refer Figure D-87.

Figure D- 87 Utah Water Research Laboratory (UWRL).

Utah Water Research Laboratory (UWRL) is a stand-alone facility located at Utah State University (USU) on the Logan River, Logan, Utah. The UWRL operates within an academic environment and collaborates with government and private sectors to address technical and societal aspects of water-related issues, including quality, quantity, distribution, and conjunctive use. This research is accomplished through providing more than 100,000 square feet of state-of-the-art laboratory, computer, and office space. The UWRL offers a wide array of hydraulic research, testing, and modeling services. UWRL is world-renowned for its expertise in water-flow modeling, offering design and construction of large physical models that are effective in improving safety, reducing construction costs, and optimizing the hydraulic performance of a wide array of existing and proposed hydraulic structures. UWRL offers a composite modeling approach that couples physical modeling with numeric modeling that is highly effective in solving a wide array of difficult hydraulic problems. UWRL has been building and testing physical scale models since its commissioning in 1965. Researchers build geometrically scaled models and utilize essential scaling parameters to accurately simulate prototype flow conditions. With both a nearby reservoir and various pumping possibilities, the UWRL can achieve modeling flow rates up to 100 cfs and physical modeling footprints that exceed 6,000 ft^2.

UWRL offers design, and construction of large-scale physical models, along with CFD computer modeling to improve safety, reduce construction costs, and optimize the efficiency of a wide array of hydraulic structures. Various types of physical models have been constructed at the UWRL. It includes: spillways of all types; energy dissipation / outlet structures; diversion structures; intake structures; fish barriers and fish passage structures; river channels; pumping plants; pipe simulations; erosion and sedimentation studies; and miscellaneous scale models of other hydraulic appurtenances. Advanced flow measurement equipment and video and photographic equipment are used to document the moving water under the modeled flow conditions to be compared with prototype conditions, providing a valuable tool for quality control.

UWRL also conducts numerical and composite/hybrid modeling including CFD studies for many projects. It combines the benefits of physical modeling with the flexibility of numerical modeling to create a valuable tool for engineers designing new hydraulic structures or improving existing ones. Numerical modeling offers insight that a physical model alone cannot offer due to the many visualization and plotting tools built into commercial CFD solvers. Numerical models are used to determine pressure, velocity, flow depth, and streamlines at any and all locations in the flow domain. Coupling of CFD model with a physical model allows the physical model to be built larger by using CFD to predict inflow boundary conditions for the physical model. CFD models are used to test rating curves of spillways prior to being built physically saving crest design iterations in the physical model and reducing the cost of the model study. CFD models are used prior to the physical model as part of the design process by modeling small portion of the hydraulic structure to look for possible problems before the physical model is constructed. CFD models are also built at full scale removing any scaling effects that can be associated with physical models.

Iowa Institute of Hydraulic Research (IIHR) - Refer Figure D-88.

Figure D- 88 Iowa Institute of Hydraulic Research (IIHR)

IIHR-Hydroscience& Engineering is a world-renowned center for education, research, and public service focusing on hydraulic engineering and fluid mechanics. Based in the C. Maxwell Stanley Hydraulics Laboratory, IIHR, a unit of the University of Iowa, is one of the nation's premier and oldest fluids research and engineering laboratories. IIHR has committed to maintaining and enhancing its experimental and field-measurement facilities while pioneering efforts in high-speed computational analysis and simulation of complex flow phenomena. IIHR maintains state-of-the-art capabilities for computational modeling, laboratory experiments, and field measurements. This permits varying, yet complementary, approaches for investigation and solution of a wide range of problems in hydroscience and engineering. Research facilities of IIHR include a variety of hydraulics flumes, wind tunnels, a ship towing tank, an ice laboratory for winter environment, a mobile hydrometeorology laboratory, and advanced equipment for laboratory and field research, such as PIV, LDV, radars and Lidars.

Since its inception, IIHR has published hundreds of research and investigation reports that have been referenced by scientists and engineers involved in water resources engineering worldwide. The focus areas of IIHR activity are as follows.

Fluid dynamics: turbulent shear flows; vortex dynamics; ship hydrodynamics; cardiovascular fluid dynamics; computational fluid dynamics.

Environmental Fluid Dynamics: river hydraulics; computational hydraulics; hydraulic structures; bio-remediation; water quality dynamics; air-water exchange processes; ice-related river hydraulics; ice mechanics; winter highway maintenance; ice modeling.

Air and Water Resources: air pollution; hydrometeorology; hydroclimatology; hydrogeology; hydrology; remote sensing; water resources. IIHR's capability in hydraulic modeling is briefly described as follows.

Physical Modeling: IIHR has perfected through decades of experience developing innovative designs and supporting applied hydraulic research. IIHR has the demonstrated expertise, experience, facilities, and equipment to build a precision model of any hydraulic structure.

CFD Modeling: IIHR is leading a worldwide revolution in the development and application of computational fluid dynamics (CFD), in support of simulation-based design (SBD). Researchers at IIHR use SBD - a sort of virtual reality, supported by model-scale experiments - to develop efficient new ways to design and test engineering designs and prototypes. Computer simulations at IIHR are complemented by model-scale physical experiments conducted at the institute‘s facilities that support the design and construction of large models. This combination of computer simulation and experimental modeling puts IIHR at the cutting edge of research in fish passage design, total dissolved gas prediction in rivers, and ship hydrodynamics.

Computational Modeling: IIHR researchers use simulation-based design — a sort of virtual reality, supported by model-scale experiments — to develop efficient new ways to design and test engineering designs and prototypes. The fully three-dimensional computer simulations run parallel to physical testing in IIHR‘s physical modeling facilities to provide high-quality, validated results.

Fully 3-D Computer Simulations: IIHR has developed fully 3-D computer simulations based on FLUENT, with several add-on functions she wrote herself. Researchers can run these computer simulations full-scale, using actual particle sizes and densities.

Complementary Physical and Computational Modeling: As an example, IIHR‘s applied its expertise to assess river hydraulics and complex flow patterns in the Ohio River near an 1800MW coal-fired power plant. The project goal was to assess the feasibility of the installation of an array of submerged screens in the cooling water intake forebay to meet regulatory requirements for fish impingement and entrainment. IIHR built a 1:24 scale physical model that included a portion of the Ohio River, the intake forebay, Units 1 and 2, and the cooling water discharge channel. IIHR calibrated the model performance to field data for validation and investigated scenarios to reduce sedimentation potential in the forebay area. IIHR recommended mitigation measures to reduce sedimentation, making installation of the submerged screens more feasible, reducing the frequency of dredging at the site, and resulting in cost savings. IIHR also built a physical model and conducted computer simulations to test the screens as well as mitigation strategies to reduce a sediment problem.

Alden Research Laboratory (ARL) - Refer Figure D-89.

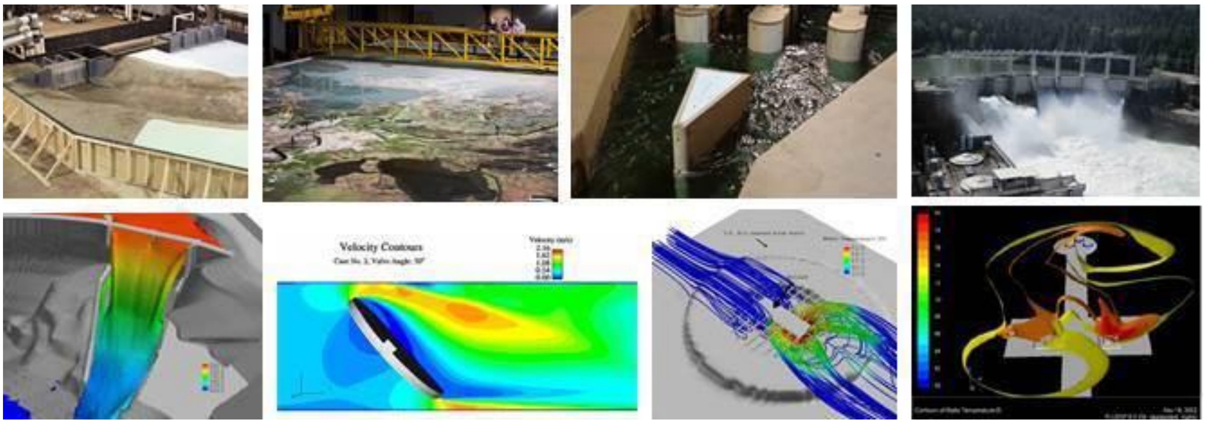


Figure D- 89 Alden Research Laboratory (ARL).

Alden Research Laboratory (Alden) in Holden, Massachusetts was founded in 1894 as part of Worcester Polytechnic Institute. It is the oldest continuously operating hydraulic laboratory in the United States. In 1986, Alden became a private company. Since its inception Alden has been conducting hydraulic model studies to solve flow problems for utilities, engineering firms and makers of hydraulic equipment. Alden has combined over a century long history of solving flow problems using physical scale models with state-of-the-art CFD software to offer the most effective method for fluid flow analysis.

ARL has maintained extensive hydraulic modeling experience in investigations of hydraulic structures, spillways & discharge systems, intake, discharge and tunnel structures, river hydraulics, and sedimentation and erosion etc. Alden maintains broad knowledge for recognizing best method for solving a problem and the most cost-effective solution based on physical and computer simulation models for conceptual design, detailed design, biological/fisheries studies, analytical modeling, CFD modeling, field measurements, physical modeling, precision flow meter calibrations, and field testing.

Alden provides services in the areas including: hydropower (control structures; turbine intakes; navigation; sedimentation; and ice issues); hydraulics of pumping stations, canal and tunnels, and lakes/harbors/coastal; flow measurement; fluid mechanic equipment; fish ladders and lifts; and air & gas flow modeling etc.

Hydraulic Structures: Alden provides hydraulic and structural design and engineering services such as analysis, design, rehabilitation, repair, and/or construction project. Alden is specialized in:

- Hydraulic structure layout, sizing, and loading.
- Hydraulic modeling (computational 1-D, 2-D and 3-D, and physical).
- Scour analysis.
- Structural evaluation and assessment.
- Detailed structural design.
- Structural rehabilitation and repair.
- 3-D structural modeling and finite element analysis.
- Analysis, design, and rehabilitation of hydraulic gates.
- Plans and specifications.
- Engineering services during construction.

Hydraulic Modeling and Consulting: Alden conducts study and design optimization for projects such as intake and discharge structures, culverts, flood walls and flood protection, dams, spillways, coastal flooding, sedimentation, erosion, canals and river mechanics. Alden addresses problems related to civil engineering hydraulics with a combination of scaled physical modeling and testing and CFD modeling.

Physical Modeling: Alden has many years of experience in developing physical models for simulation of the prototype flow system. Physical models are used for studying flow patterns, pressure losses, constituent transport and reaction rates, forces on structures and other phenomena of interest for a given design.

Numerical Modeling: Alden applies advanced 1-D, 2-D and 3-D numerical modeling tools to evaluate many complex flow systems that used to require laboratory physical modeling or extensive field testing programs to study. CFD models are used to predict the performance of most flow systems and include the simulation of multiphase flow (gas/water, water/particulate and aeration), chemical species transport, reaction kinetics and mass transfer. Modeling of flow turbulence and energy dissipation are included in these models and facilitate prediction of complex hydraulic, chemical and thermal mixing processes. Alden has extensive experience in the analysis of scour and sediment deposition under a variety of conditions using 3-D modeling to predict erosion and sedimentation in various riverine environments. Alden maintains a broad range of three dimensional modeling tools including the commercial software packages such as Fluent, Flow3D and FIDAP. Alden applies these CFD software programs to carry out the investigations.

Modeling Facilities: Alden maintains unique modeling facilities and sophisticated data acquisition systems for each study. Alden in Holden, Massachusetts houses approximately twenty buildings on thirty acres and is equipped with more than 125,000 square feet of enclosed space, flow supplies and control systems for conducting hydraulic modeling. Two nearby ponds ensure the reliable, year-round water source so critical to physical modeling. In addition, the Alden facility in Everett, Washington provides 20,000 square feet of modeling and testing space. Alden‘s dedicated modeling facilities and equipment includes:

- Large scale river modeling facility.
- Large flow spillway modeling facility.
- Pump intake modeling facility.
- High flow pump capacity.
- Sumps in all model buildings.
- Gas flow modeling facility.
- Dedicated computing equipment and software for numerical and computational fluid dynamic modeling.

Some examples demonstrating the capability and experience of Alden are given as follows.

CFD Modeling of Spillways and Discharge Structures

Smith Mountain Dam is a double arch concrete dam which impounds flow from the Roanoke and Blackwater Rivers. The principle concern of the project was the trajectory of the overtopping flow and its impacts the catch chute. Alden conducted a model study of the spillway flip-bucket for overtopping flow during the probable maximum flood. A 3-D CFD model of a portion of the reservoir and the dam was developed using Flow3D simulation software. The results of the CFD model were compared to the results of a physical model study (1:60 scale) conducted at Alden to validate the model (Figure D-90). The simulated characteristics of the free falling water were compared to photographs of the physical model and flow features such as contractions, surface waves, back eddies and the impact point on the catch chute was quantitatively verified. Further, a comparison of the discharge rating curve as calculated from the physical model and CFD results was made. The CFD predicted discharge for a given stage was within 5 percent of the measured values. Some images of CFD spillway simulations are shown as follows. The CFD code was also used to study air entrainment and dissolved gas concentrations in the downstream reaches.

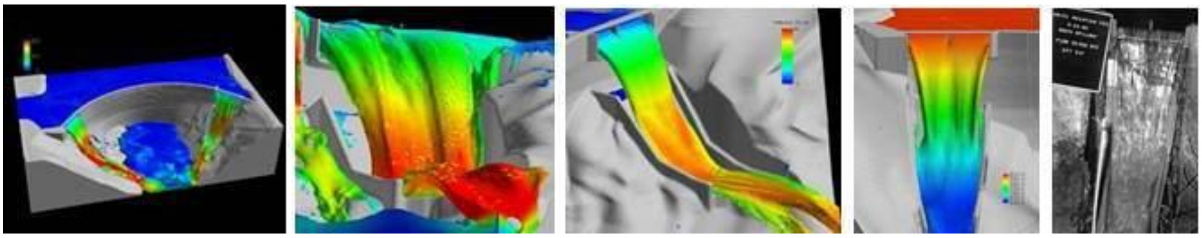


Figure D- 90 some images of the Smith Mountain Dam spillway CFD simulation.

CFD Modeling of Forebay/Intake Approach Flows

Alden has conducted several CFD model studies of the approach flow to intake structures and within powerhouse forebays. Many of these studies have been in support of fish guidance and passage issue. As an example, Alden conducted multiple CFD analyses of the Hadley Falls Hydroelectric Project including detailed flow patterns within the forebay area, across the face of the dam, through a bypass canal and through the intake structures. The commercial codes Fluent and Flow3D were used to simulate this intake. CFD simulation results were quantitatively compared with results of a physical model study conducted at Alden previously to verify that the velocity magnitudes and direction were properly simulated at the entrance to the trash racks (sample comparison plot of flow direction shown in Figure D-91).

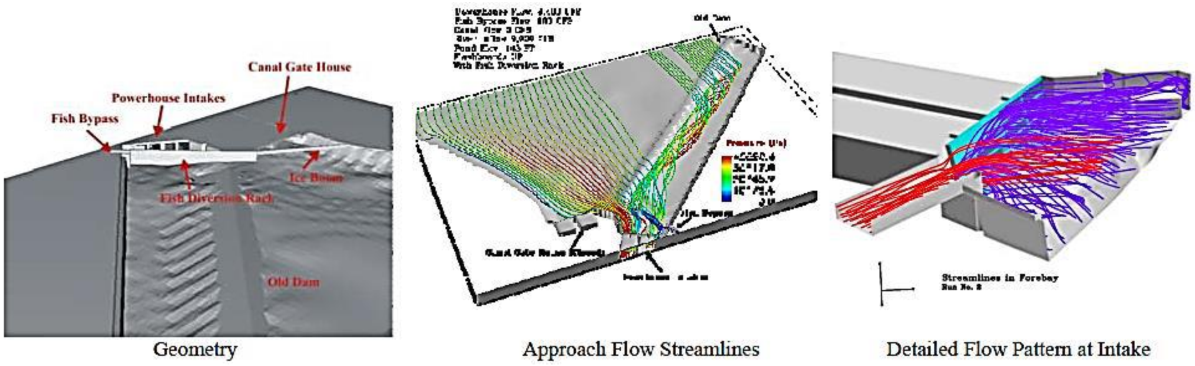


Figure D- 91 CFD Model of Hadley Falls Forebay and Intake Structure.

CFD Modeling of Tunnel Flows and Flow Control Devices

Alden has conducted numerous studies involving flow through tunnels and regulating structures based on a CFD analysis using Fluent to predict the discharge coefficient associated with a butterfly-type regulating valve in a large diameter tunnel for a hydroelectric project as shown in Figure D-92. Using Fluent model Alder also performs CFD analyses of the control structures, potential for cavitation within tunnels and flow for various gate positions.

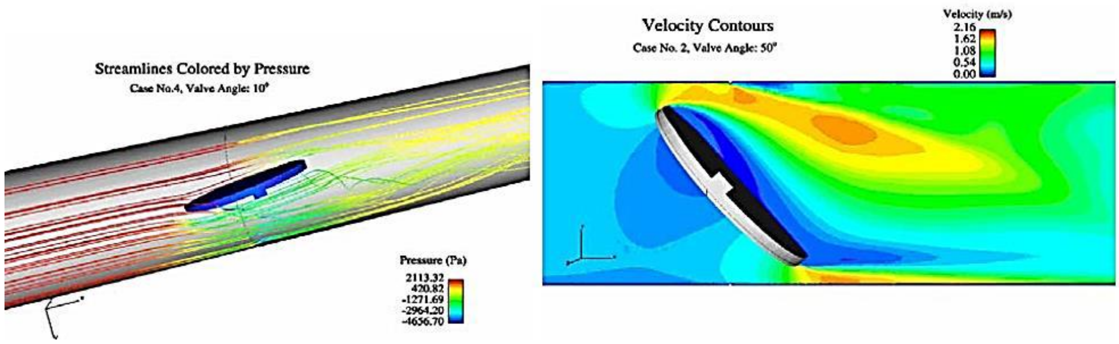


Figure D- 92 CFD Simulations of Flow through a Tunnel with a Regulating Structure.

Laboratory of Hydraulic Constructions – Lausanne (LCH), Switzerland- Refer Figure D-93.

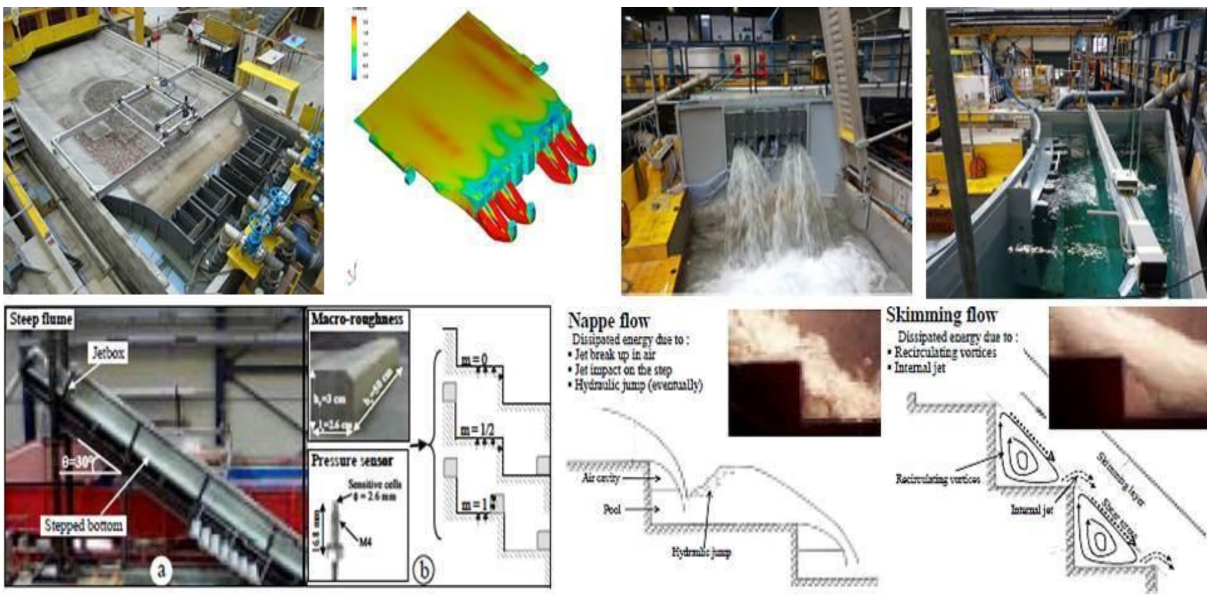


Figure D- 93.Laboratory of Hydraulic Constructions – Lausanne (LCH).

The LCH activities comprise education, research and services in the fields of both applied hydraulics and design of structures. Aspects of hydraulic structures related to flood control, hydropower plants, water intakes, dams, etc. are studied. These investigations are mainly carried out using scale models, numerical simulations, and the combination of both. The research projects at the LCH are focusing on the following areas:

- Dynamic behavior of earth and rock fill dams under seismic loading, finite-element modeling of unbounded media.
- Overflow dams, high velocity flow on steep slope over macro-roughness.
- Overflow and fuse plug river banks, influence of lateral spill of water on the sediment transport capacity of a river channel.
- Sediment transport in steep Alpine rivers, erosion and scouring in bends, influence of roughness of bank protection works.
- Shore erosion protection measures, oil spill retention system.
- Dynamic water pressure in rock fissures due to high velocity water jets, modeling of scouring problems in fissured media downstream of dams.
- Reservoir management, surface erosion of natural catchments, density and turbidity currents, silting-up of reservoirs, flushing.
- Natural risk evaluation, design of extreme floods, influence of land development, debris flow, climate change impacts, sediment transport in rivers and mobile bed processes.
- Hydro-informatics, numerical simulation of water flow in complex hydraulic systems, analysis and numerical modeling of runoff in rural and urban areas, surface roughness, and water management at watershed scale.

HR Wallingford, UK- Refer Figure D-94.

Figure D- 94 HR Wallingford

HR Wallingford is an independent civil engineering and environmental hydraulics organization specialized in engineering hydraulics. It was originally established, in 1951, in Wallingford as the Hydraulics Research Station and changed name to HR Wallingford Limited in 1991. The focus of this organization is increasingly on climate change, adaptation and resilience to extreme events. They have assessed the impacts of climate change on water resources (surface water and groundwater) in over 70 UK river basins, as well as major basins overseas. Their specialists in the field of river engineering and drainage use a range of methods to assess and optimize hydraulic structures, from desk assessments and CFD studies through to large scale physical models. The capability and facility of HR Wallingford are briefly described as follows.

Modeling hydraulic structures: HR Wallingford has facilities to undertake free-standing scale model studies. Despite advances in computational fluid dynamics (CFD), physical models are used to investigate the performance of pumping stations and of complex and non-standard structures such as spillways; curved or gated weirs; water and river intakes; drop structures; outfalls; and bridge piers etc.

Physical and numerical modeling for dams and reservoirs: The state-of-the-art physical modeling facilities are used to optimize structure designs at an early stage, preventing costly operational issues later. The complimentary numerical modeling systems ensure a solution for any complex hydraulic design problem.

- Physical modeling of flow pattern, scour erosion, and wave impact for dams, spillways, gates, channels, bridge piers and more.
- Use of the INFOWORKS hydraulic modeling suite of software to model 1-D or 2-D flow.
- Use of the RESSASS software to model sedimentation processes.
- Use of HR BREACH (original and zoned) and AREBA software to model embankment breach development.
- Use of GIS software to model dam-related flooding downstream.

Reservoir sedimentation and operation: Sedimentation causes an estimated one per cent reduction in the total capacity of all reservoirs worldwide annually. Sediments also block intakes in reservoirs and damage tunnels and turbines. HR Wallingford has undertaken research and consultancy studies in reservoir sedimentation over many years. HR Wallingford provides services in sediment measurement techniques, reservoir surveys and their analysis, sediment transport processes in river basins and the application of numerical models in reservoir sedimentation studies. Numerical models are used to predict future reservoir storage and test operational changes. HR Wallingford provides expert advice, desk studies, field monitoring, surveying and its analysis, numerical modeling and training.

Dam break assessment and flood mapping: Dam breaks or embankment (levee/dike) breaches can have enormous cost and loss of life consequences. HR Wallingford provides the following services:

- Scenario assessment and potential risk identification.
- Use of simplified breach software (AREBA) for simple embankments within system models.
- Use of detailed software (HR Breach / EMBREA) to predict breach growth through complex embankment structures.
- Sale of breach software (EMBREA) as stand-alone, or as part of consultancy.
- General introduction to breach modeling training course.
- Use of InfoWorks-RS or other flood modeling software to simulate the reservoir failure.
- Flood routing and inundation mapping (1-D, integrated 1-D & 2-D, 2-D).
- Associated risk assessments and emergency planning.
- Agent-based loss of life modeling using life safety models.
- Development of emergency management plans.

Physical modeling: The physical modeling facilities extend to approximately 14,400 m² and are housed in three purpose built halls. The physical modeling facilities include:

- Hydraulic structure and river floodplain modeling area.
- General purpose flow flume with certified volumetric measurement capability.
- Wave basins - both separate and linked units capable of extension to 2,400 m².
- Wave-current or current only basin with unidirectional or bidirectional current discharges.
- Wave flumes equipped with wave absorbing paddles to minimize wave reflections within the flumes.
- Special facilities for tsunami generation, flood protection product testing, air in pipelines and aircraft ditching studies.

Delft Hydraulics Laboratory / Hydraulic Engineering Laboratory – TU Delft, Netherlands - Refer Figure D-95.

Figure D- 95 Delft Hydraulics Laboratory / Hydraulic Engineering Laboratory – TU Delft.

The Delft Hydraulics Laboratory was established in 1927. It is specialized in applied scientific research in the fields of hydraulic structures and hydrology, provides basic data for the design of hydraulic structures, river- and coastal engineering works, supplies design criteria for industrial circulations, computational hydrodynamics, in-situ surveys and supports consulting works etc. The studied flows are mainly characterized by the presence of a free surface. Shallow-water turbulence, sediment transport, and surface wave motion are some examples of research fields. Their core activities are managing, procuring and maintaining the test facilities and equipment in the laboratory and for field measurements. The laboratory organization includes a major field measurement section where field measurement equipment for courses and research is developed and prepared, and assistance is given for field measurement campaigns. Their areas of specialization include:

- Rivers and navigation.
- Locks, weirs and sluices (cavitation and vibration).
- Hydrodynamics and morphology.
- Offshore and maritime structures.
- Harbors and coastal zones.
- Pumps and industrial circulation.
- Density currents and estuarine hydraulics.
- Sediment transportation and multiphase flow.

- Dredging technology.
- System approach.
- Measuring methods and instrumentation etc.

The facilities maintained by the laboratory include:

- Mechanical and electronic engineering workshops.
- Instrumentation laboratories.
- Photography laboratory.
- Sediment sieving and mixing facility.
- Sediment analysis laboratories.
- Wind-wave and current flumes.
- Pump testing equipment.
- Calibration rigs for discharge meters.
- Density current provisions with warm water and salt water supply.

17. Software Systems and Packages for Computational Hydraulics

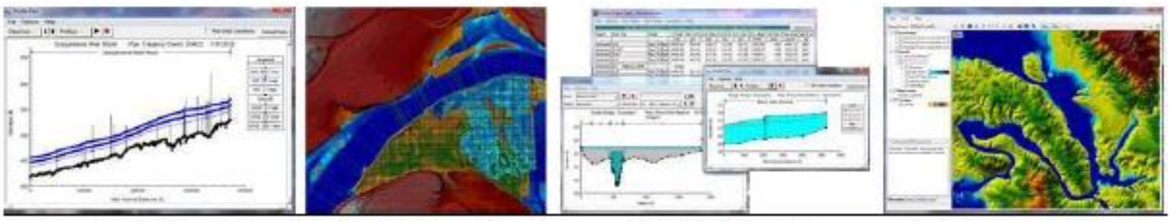
There are several approaches to numerical modeling such as 1-D, 2-D and 3-D methods and a combination of these methods. A full 3-D method based on solving mathematical equations of fluid dynamics is commonly referred to as CFD (computational fluid dynamics) method. The most common CFD computer programs solve the Reynolds-averaged Navier-Stokes equations using a variety of computational techniques. The evolution of commercial CFD software has been occurring continuously due to the rapid advancement in computing technology, high performance computing, data mining and visualization, and cloud computing, etc. Any hydrodynamic modeler who is entrusted with a task to perform a numerical modeling task of a significant hydraulic study should be expected to have an expert level knowledge of the theoretical basis of the numerical methods and hydrodynamic modeling with several years of experience in the application of a computer software package. It is important that computer models are validated [86, 91]. Expert-level elicitation is needed to proof-test the analyses results. For performance and safety evaluation of dams, numerical simulations should not be accepted blindly. Any hydraulic simulation software is a powerful analysis tool, but it should be used with care. Many computer programs suitable for performing general hydraulic analyses were described in King‘s Handbook of Hydraulics (seventh edition). They can also be found in many other sources [133-137]. Due to the rapid advancement made in software engineering related to pre- and post-processing, data mining, and data visualization, these computer programs and software packages have evolved, enhanced, and become more powerful with useful tools for assessing hydraulic issues related to dam safety. They are often used to complement physical model studies or applied in place of physical model studies.

Some computer programs and software packages that can be applied to perform dam safety related hydraulic analysis are briefly described as follows. It includes computer programs developed by the government agencies, hydraulic research institutions, and private developers.

- HEC-RAS for steady and unsteady flow water surface profile calculations.
- FLO-2D for dynamic flood routing hydrologic and hydraulic model.

- SMS provides tools for every phase of a hydraulic simulation.
- FEMSWS-2DH is a 2-D depth-integrated finite element flow modeling system.
- MIKE-21 is an unsteady hydrodynamic model for rivers, estuaries, bays and flood plains.
- MIKE 3 for modeling 3-D free-surface flows and sediment and water quality processes.
- CE-QUAL-W2 is a 2-D laterally-averaged hydrodynamic and water quality model.
- CE-QUAL-R1 is a 1-D width-averaged hydrodynamic and water quality model.
- WQRRS is 1-D model for flow and water quality in rivers and reservoirs.
- DYRESM is a 1-D dynamic simulation model for small to medium-sized impoundments.
- FLOW-3D is a CFD software for solving free-surface hydraulic and fluid flow problems.
- ANSYS FLUENT is a general purpose computation fluid dynamics software suite.
- FENFLOSS is a general purpose computation fluid dynamics software suite.
- STAR-CCM+ is a CFD software tool for coupled multiphysics simulation.
- PHOENICS is a general purpose computation fluid dynamics software suite.
- Open TELEMAC-MASCARET is an integrated suite of solvers for free-surface flows.

HEC-RAS- Refer Figure D-96.



Steady Flow Water Surface Profiles 1-D/ 2-D Unsteady Flow Simulation Graphics and Reporting Mapping of water surface profile

Figure D- 96 HEC-RAS

HEC-RAS supports steady and unsteady flow water surface profile calculations; combined 1-D and 2-D hydrodynamics; sediment transport/mobile bed computations; water temperature analysis and water quality analyses (nutrient transport and fate); and spatial mapping of many computed parameters (such as depth, water surface elevation, and velocity etc.) for a full network of natural and constructed channels. HEC-RAS is an integrated system of software, designed for interactive use in a multi-tasking, multi-user network environment. The system is comprised of a graphical use interface (GUI), separate hydraulic analysis components, data storage and management capabilities, graphic and reporting facilities. The system contains four one-dimensional river analysis components for: (1) steady flow water surface profile computations; (2) unsteady flow simulation (1-D and 2-D hydrodynamics solving full Saint Venant equations or Diffusion Wave equations); (3) movable boundary sediment transport computations; and (4) water quality analysis. A key feature of the HEC-RAS 2-D implementation is the use of a high resolution subgrid model where the computation cells do not have flat bottoms but rather are represented by geometric and hydraulic property tables based upon the underlying terrain. This allows for utilization of larger computational grid cell sizes while retaining the resolution of the topographic terrain which provides for reduced computational time. This software system was developed at the Hydrologic Engineering Center (HEC), which is a division of the Institute for Water Resources (IWR), U.S. Army Corps of Engineers. The following is a description of the key capabilities of HEC-RAS including: user interface; hydraulic analysis components; data Storage and management; graphics and reporting; and RAS mapper.

User Interface.

The user interacts with HEC-RAS through a graphical user interface (GUI). The interface provides the following functions: file management; data entry and editing; hydraulic analyses; tabulation and graphical displays of input and output data; inundation mapping and animations of water propagation; reporting facilities; and context sensitive help.

Hydraulic Analysis Components.

The HEC-RAS system contains several river analysis components for: (1) steady flow water surface profile computations; (2) 1-D and 2-D unsteady flow simulation; (3) movable boundary sediment transport computations; and (4) water quality analysis. All four components use a common geometric data representation and common geometric and hydraulic computation routines.

Steady Flow Water Surface Profiles. The system can handle a full network of channels, a dendritic system, or a single river reach. This component is capable of modeling subcritical, supercritical, and mixed flow regimes water surface profiles. The basic computational procedure is based on the solution of the one-dimensional energy equation. Energy losses are evaluated by friction (Manning's equation) and contraction/expansion (coefficient multiplied by the change in velocity head). The momentum equation may be used in situations where the water surface profile is rapidly varied. These situations include mixed flow regime calculations (i.e., hydraulic jumps), hydraulics of bridges, and evaluating profiles at river confluences (stream junctions).

One- and Two-Dimensional Unsteady Flow Simulation.

This component is capable of simulating one-dimensional; two-dimensional; and combined one- and two-dimensional unsteady flow through a full network of open channels, floodplains, and alluvial fans. The unsteady flow component can be used to performed subcritical, supercritical, and mixed flow regime (subcritical, supercritical, hydraulic jumps, and draw -downs) calculations in the unsteady flow computations module. Special features of the unsteady flow component include: extensive hydraulic structure capabilities dam-break analysis; levee breaching and overtopping; pumping stations; navigation dam operations; pressurized pipe systems; automated calibration features; user defined rules; and combined 1-D and 2-D unsteady flow modeling.

Steady Transport/Movable Boundary Computations.

This component of the system is intended for the simulation of one-dimensional sediment transport/movable boundary calculations resulting from scour and deposition over moderate time periods (typically years, although applications to single flood events are possible). The sediment transport potential is computed by grain size fraction, thereby allowing the simulation of hydraulic sorting and armoring. Major features include the ability to model a full network of streams, channel dredging, various levee and encroachment alternatives, and the use of several different sediment transport equations. The model is designed to simulate long-term trends of scour and deposition in a stream channel that might result from modifying the frequency and duration of the water discharge and stage, or modifying the channel geometry. This system can be used to evaluate deposition in reservoirs, design channel contractions required to maintain navigation depths, predict the influence of dredging on the rate of deposition, estimate maximum possible scour during large flood events, and evaluate sedimentation in fixed channels.

Water Quality Analysis.

An advection-dispersion module is included in addition to the capability to model water temperature. This module uses the QUICKEST-ULTIMATE explicit numerical scheme to solve the one-dimensional advection-dispersion equation using a control volume approach with a fully implemented heat energy budget. Transport and fate of a limited set of water quality constituents is available such as Dissolved Nitrogen; Dissolved Phosphorus; Algae; Dissolved Oxygen (DO); and Carbonaceous Biological Oxygen Demand (CBOD).

Data Storage and Management.

Data storage is accomplished through the use of "flat" files (ASCII and binary), the HEC-DSS (Data Storage System), and HDF5 (Hierarchical Data Format). User input data are stored in flat files under separate categories of project, plan, geometry, steady flow, unsteady flow, quasi-steady flow, sediment data, and water quality information. Output data is predominantly stored in separate binary files. Data management is accomplished through the user interface.

Graphics and Reporting.

Graphics include X-Y plots of the river system schematic, cross-sections, profiles, rating curves, hydrographs, and inundation mapping. A three-dimensional plot of multiple cross-sections is also provided. Inundation mapping is accomplished in the Mapper portion of the software. Inundation maps can also be animated, and contain multiple background layers (terrain, aerial photography, etc.).

RAS Mapper.

It has the capability to perform inundation mapping of water surface profile results directly from HEC-RAS. Using the geometry and computed water surface profiles, inundation depth and floodplain boundary datasets are created through the RAS Mapper using a terrain model. Additional geospatial data can be generated for analysis of velocity, shear stress, stream power, ice thickness, and floodway encroachment data.

FLO-2D- Refer Figure D-97.

Figure D- 97 FLO-2D

FLO-2D is a complete dynamic flood routing hydrologic and hydraulic simulation model with many urban detail features, river and floodplain interface, sediment transport, storm drain component, mudflow and groundwater modeling. FLO-2D is a 2-D flood routing model that combines hydrology and hydraulics for inundation risk assessment and hazard mapping and for the engineering design of flood mitigation works. FLO-2D has a wide variety of applications such as flood mitigation studies, storm drain modeling, dam breach analysis, surface and ground water interactions, mud flows, and sediment transport. The flood modeling components include: hydrology (spatially variable rainfall; infiltration – impervious surfaces; and building roof runoff), channel routing (variable geometry cross sections; hydraulic structures; bridges; culverts; weirs; pumps; levees and levee breach; and sediment transport), overland components (dam and levee breach; spatially variable roughness; Froude number roughness adjustment; rill and gully flow; and mud flows), urban features (complete storm drain system; building walls and obstruction collapse; and low impact development).

The FLO-2D model was conceptualized in 1986 to predict mudflow hydraulics. The US Federal Emergency Management Agency (FEMA) supported the initial model development. FLO-2D is a widely used commercially available flood model due to its capability to simulate urban flooding in high resolution and excellent detail including the storm drain system. The hydrological component is a rainfall-runoff model, with an overland flow model that simulates the movement of the flood volume around the grid. Flow conveyed into the channel is routed using the 1-D Saint Venant wave equation. Key features of the FLO-2D model include a Graphical User Interface (GUI), Grid Developer System.

(GDS), and Mapper. These features allow the user to process and edit the grid data, graphically edit hydraulic structures, and create flood risk and flood hazard maps. The capabilities of the FLO-2D Pro Model include: full storm drain/surface water interface; channel-storm drain exchange; manhole popping; rainfall/infiltration with spatially varied data; wall and building flow obstruction; building roof runoff, downspout control, parapet walls; wall and levee breach and collapse; and building collapse. The advantages of using FLO-2D include: ability to integrate different types of geospatial data e.g., LIDAR, aerial images, shape files, and contour maps; import HEC-RAS geometry cross-sections; can be coupled with other programs such as SWMM and MODFLOW; and model storm surge and tsunami impacts.

The limitation includes: grid element represents single elevation, Manning's n value, and flow depth; and rapidly variable flow, i.e. a dam breach, is not simulated 1-D channel flow (no secondary currents, or vertical velocity distributions).

SMS Surface-water Modeling System- Refer Figure D-98.

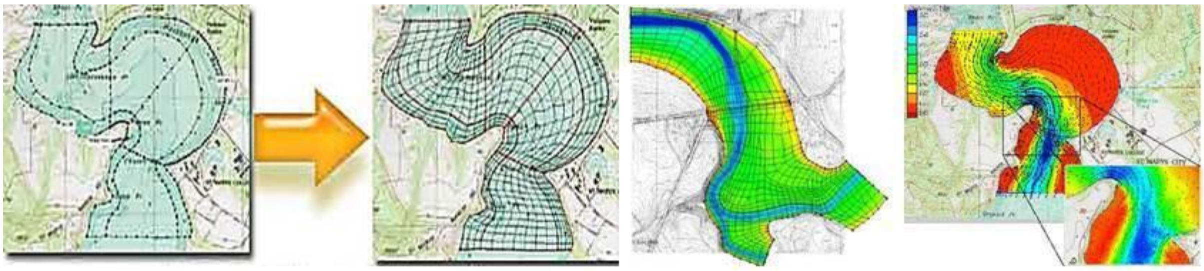


Figure D- 98 SMS Surface-water Modeling System

The SMS Surface-water Modeling System is a comprehensive package, which provides tools for every phase of a hydraulic simulation including site characterization, model development, post-processing, calibration, and visualization. SMS provides an integrated graphical environment for performing surface flow, contaminant fate/transport, and project design evaluations. The system is a comprehensive environment for one-, two-, and three-dimensional hydrodynamic modeling. A pre- and post-processor for surface water modeling and design, SMS supports many 2-D finite element, 2-D finite difference, and 3-D finite element hydraulic modeling tools. The numeric models supported by SMS compute a variety of information applicable to surface water modeling. Primary applications of the models include calculation of water surface elevations and flow velocities for shallow water flow problems, for both steady-state or dynamic conditions. Additional applications include the modeling of contaminant migration, salinity intrusion, sediment transport (scour and deposition), wave energy dispersion, wave properties (directions, magnitudes and amplitudes) and others.

River hydrodynamics can be modeled with SMS using one of several 2-D simulation models, including FESWMS, RMA2, HIVEL2D and TUFLOW. River models allow users to predict water depth and velocity in complex waterways including bays, estuaries, and river reaches. Natural and man-made conditions can be simulated in unprecedented detail using the SMS pre and post processing tools. SMS has evolved from a simplistic mesh generator to an all-inclusive tool for start-to- finish numerical modeling of surface water related hydraulic issues. The graphical interface houses independent numerical models used to conduct everything from hydrodynamic circulation patterns, to sediment transport or contaminant transport. The user may conceptualize the boundary conditions for the model domain and choose to analysis, the results from one of a dozen numerical models supported within SMS. The Lagrangian Particle Tracking Model (PTM) of SMS is a very useful feature for visualization of the simulated flow. SMS provides a complete interface for numerical models supported by the USACE-ERDC such as: ADCIRC, ADH, CGWAVE, TABS-MD (RMA2, RMA4, SED2D), TABS-MDS (aka RMA10-WES), BOUSS-2D, CMS-Flow and CMS-Wave, STWAVE, and other generic models. A comprehensive interface has also been developed for facilitating the use of the FHWA commissioned analysis package FESWMS. The TUFLOW numerical model with flood analysis, wave analysis, and hurricane analysis is also supported. SMS also includes a generic model interface, which can be used to support models which have not been officially incorporated into the system. New enhancements and developments have been continued at the Environmental Modeling Research Laboratory (EMRL) at Brigham Young University in cooperation with the U.S. Army Corps of Engineers Waterways Experiment Station (USACE-WES), and the US Federal Highway Administration (FHWA).

With the Automated Mesh/Grid Generation feature, SMS can be used to construct 2-D and 3-D finite element meshes and finite difference grids of rivers, estuaries, bays, or wetland areas. The tools include a sophisticated set of creation and editing tools to handle complex modeling situations. Several methods of finite element mesh creation are available, allowing user to create any combination of rectangular and triangular elements needed to represent model domain. Both Cartesian and boundary-fitted grid creation tools are available to allow representation of a model domain for finite difference models. The mesh/grid creation tools coupled with GIS objects, makes SMS an easy-to-use and accurate modeling system.

There are two main methods for building models in SMS, the direct approach and the conceptual modeling approach. With the direct approach, the first step is to create a mesh or grid. The model parameters, source/sink data, and boundary conditions are assigned directly to the nodestrings, nodes, and elements of the mesh. The most efficient approach for building realistic, complex models is the conceptual modeling approach. With this approach, a model is created using GIS objects, including points, arcs, and polygons. The conceptual model is constructed independently of a mesh or grid. It is a high-level description of the site including geometric features such as channels and banks, the boundary of the domain to be modeled, flow rates and water surface elevations of boundary conditions, and material zones with material properties such as Manning's n value. Once the conceptual model is complete, a mesh or grid network is automatically constructed to fit the conceptual model, and the model data are converted from the conceptual model to the elements and nodes of the mesh network.

SMS allows users to take advantage of all types of GIS data available for hydraulic modeling. The Map module of SMS includes a complete set of tools for importing, creating, and manipulating GIS vector and raster data. SMS can work with user‘s GIS data effectively with or without ArcGIS. A few of the useful tools in SMS include:

- Robust algorithms to handle large data sets (such as bathymetry data collected by LIDAR survey) with speed and accuracy;
- Images (such as TIFF and JPEG) can be geo-referenced, joined, and clipped;
- Use TIFF or JPEG images to guide on-screen digitizing and to enhance presentation;
- Boundary conditions and material properties from data layers can be assigned by using GIS overlay operations;

- Coordinate System Conversions - Convert data between geographic and planar coordinate systems;
- Control mesh/grid density and type by assigning properties to simple GIS objects; and
- Create observation points/cross sections for review and calibration of model output.

FEMSWS-2DH- Refer Figure D-99

Figure D- 99 FEMSWS-2DH

FESWMS-2DH is a finite-element surface-water modeling system for two-dimensional flow in the horizontal plane. It is a modular set of computer programs that simulates two-dimensional, depth-integrated, surface-water flows. FESWMS-2DH has been developed to analyze flow at bridge crossings where complicated hydraulic conditions exist, and may be applied to many types of steady or unsteady flow problems. FESWMS has capabilities which are well-suited for modeling weirs, piers, culverts, and drop inlets, shallow rivers, harbor, flood plains, estuaries, and coastal seas in which flows are essentially two-dimensional in the horizontal plane. FESWMS-2DH is short for the Finite Element Surface Water Modeling System: 2-Dimensional Flow in a Horizontal Plane. FESWMS-2DH consists of an input data preparation program (DINMOD), flow analysis model (FLOMOD), simulation output analysis program (ANOMOD), and graphics conversion program (HPPLOT). FESWMS was developed by D. C. Froehlich with the support of the Federal Highway Administration (FHWA). FESWMS-2DH is written in Fortran 77 and the code has been used on UNIX-based computers. SMS can be used to define all boundary conditions and governing material properties. The output solution file can be read into SMS and viewed using post-processing tools.

In 1994, FHWA collaborated with the Engineering Computer Graphics Laboratory at Brigham Young University for enhancement of FESWMS-2DH. Later, the lab was reorganized to form the Environmental Modeling Research Laboratory (EMRL). Work previously completed by EMRL and the USACE-WES was the basis of the FHWA-EMRL effort to create an interface to the FHWA's FESWMS-2DH surface water modeling system. That interface and corresponding tools were made available as part of the Surface-Water Modeling System (SMS) which was released in 1995.

The interface to SMS is divided into modules to simplify its use. Sets of random data points are grouped into scattered data sets" and are used in the Scattered Data Module. Finite element networks that consist of nodes, elements, element properties, and boundary conditions are created and manipulated through use of the Mesh Module. Input data files are also created through use of the Mesh Module. The Mesh Module is used to create finite element networks that consist of many interrelated nodes and elements. The Scattered Data Module is typically used to provide topographic information to the nodes in the finite element network. The system also allows the user to create and edit networks that have multidimensional attributes. SMS includes a model checker that can compare aspects of a finite element network and data file with the capabilities of a selected model. If any errors are identified, they are reported to the users as potential errors. The quality" of the mesh can also be evaluated through the use of a mesh quality tool. If poorly constructed elements are found within the network, they are graphically identified. The user may or may not correct the problem by using one of the numerous interactive editing tools within SMS to manually edit and modify the finite element network. Once a network is constructed, model-dependent material properties and boundary conditions are easily assigned.

SMS also includes a number of algorithms that can automate the creation of finite element networks. These algorithms include triangulation, patch-based network creation, adaptive tessellation (which forms a finite element network with a mosaic pattern), and finite element network and model creation using Geographical Information System (GIS) features and concepts. The GIS capabilities and concepts use arcs, polygons, and coverage that can easily be used to rapidly create and modify finite element networks. The tools included within SMS make it possible to apply data-intensive, state-of-the-art models such as FESWMS-2DH to numerically simulate complex surface-water flows. The simulations are cost-effective and provide realistic solutions to complex problems commonly encountered by hydraulic engineers. SMS incorporates user-friendly tools that can help hydraulic engineers apply a variety of types of surface-water models. The system makes it possible to solve a problem using multiple approaches or models and thereby gain a fuller understanding of the conditions.

MIKE-21- Refer Figure D-100

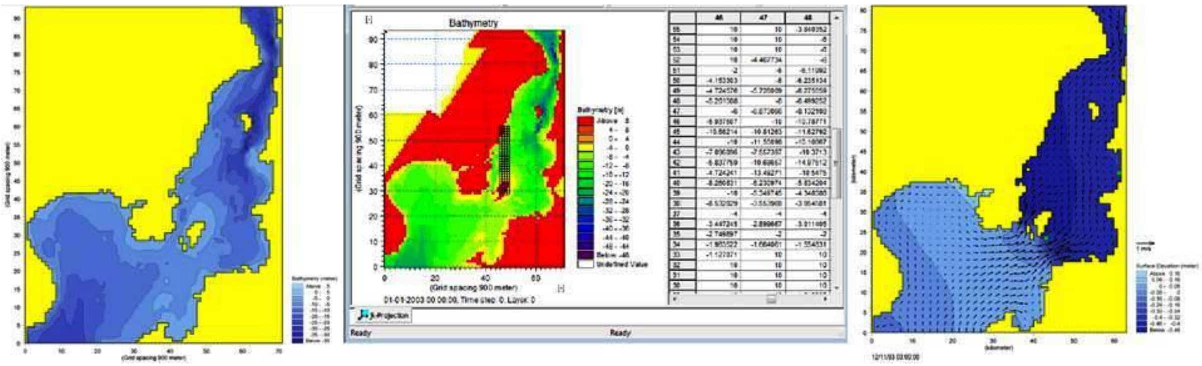


Figure D- 100 MIKE-21

MIKE-21 is developed and maintained by DHI. It is a versatile tool for coastal modeling for:

- Design assessment for coastal and offshore structures.
- Optimization of port layouts and coastal protection measures.
- Cooling water, desalination and recirculation analysis.
- Optimization of coastal outfalls.
- Environmental impact assessment of marine infrastructures.
- Ecological modeling including optimization of aquaculture systems.
- Optimization of renewable energy systems.
- Water forecast for safe marine operations and navigation.
- Coastal flooding and storm surge warnings.

- Modeling of inland river, flooding and overland flow.
- MIKE 21 includes the following modules specifically for wave modeling.
- SW - Spectral Waves: This is a spectral wind-wave model that simulates the growth, decay and transformation of wind-generated waves and swells.
- BW - Boussinesq Waves: This is a tool for studies and analyses of wave disturbance in ports, harbors and coastal areas. It includes full surf and swash zone dynamics.
- MA – Mooring Analysis: This module simulates the motions of single or multiple vessels subject to winds, waves and currents. It also calculates the forces in fenders and mooring lines.

MIKE 21 includes the following modules specifically for sediment transport and water quality modeling.

- ST - Sand Transport: This is an advanced sand transport model with several formulations for current as well as current-wave generated transport, including 3D description of sediment transport rates. It is, for example, used for morphological optimization of port layouts, impact of shore protection schemes and stability of tidal inlets.
- MT - Mud Transport: This is a combined multi-fraction and multi-layered model that describes erosion, transport and deposition of mud or mixtures of sand and mud.
- PT - Particle Tracking: This module simulates transport and fate of dissolved and suspended substances, including sediments.
- SM - Shoreline Morphology: This module combines detailed 2-D modeling of currents and waves with a constrained morphological model, making it possible to undertake fast, stable and robust modeling of shoreline evolution in 2-D environments.
- OS - Oil Spill: This module simulates the spreading and weathering of hydrocarbons and is used for oil spill modeling.

MIKE Eco Lab - Ecological Modeling: This is a complete numerical laboratory for water quality and ecological modeling.

MIKE 21 is modular. It includes a wide range of modules, allowing users to create their own tailored modeling framework for their coastal and marine studies.

- PP - Preprocessing and Post-processing: With this module the tasks of data input, analysis and presentation of simulation results are simplified.
- HD – Hydrodynamics: This module simulates water level variations and flows in response to a variety of forcing functions.
- AD - Advection-Dispersion: This simulates the transport, dispersion and decay of dissolved or suspended substances. It is typically used in cooling water and wastewater discharge studies.
- Coupled Modeling: The FM (Flexible Mesh) series include a powerful, integrated system which combines wave, flow and sediment transport models into a fully dynamic morphological model.
- MIKE 21 comprises the following simulation engines:
- Single Grid, which is a classic rectilinear model that is easy to set up and with easy I/O exchange.

- Multiple Grids, which is a dynamically nested rectilinear model with the ability to focus the grid resolution.
- Flexible Mesh, which allows maximum flexibility for adapting grid resolution of the model domain.

MIKE 21 also includes a number of tools to optimize the work.

- Global tide data and tools for tidal analysis and prediction.
- Climate Change Editor.
- Cyclone wind generation and wind generation from pressure maps.
- Advanced mesh and grid generators and editors.
- Advanced tools for generation of graphical output.

An interface for reading and modifying files in MIKE 21's internal, binary format.

All Flexible Mesh and Single Grid engines support parallel processing. The Flexible Mesh (FM) engines show excellent performance when parallel processing is undertaken - also on a large number of computational cores. On multicore Windows computers, parallelization is menu-driven and straightforward.

MIKE 3- Refer Figure D-101

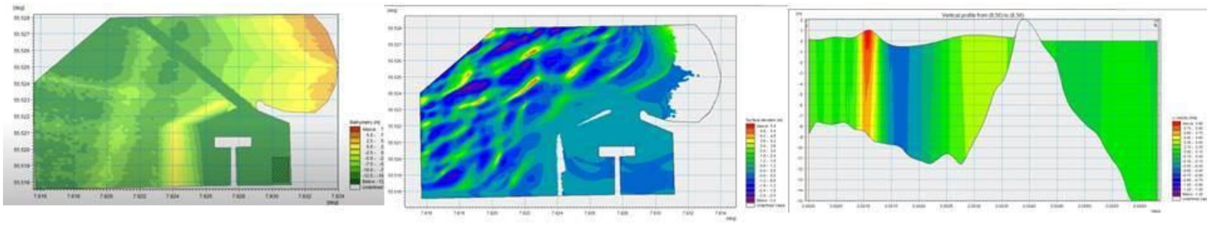


Figure D- 101 MIKE 3

MIKE 3 is developed and maintained by DHI. It provides the simulation tools for modeling 3-D free surface flows and associated sediment or water quality processes. MIKE 3 is a widely recognized simulation model for environmental and ecological studies. MIKE 3 builds on the same solid technology as MIKE 21 and is ideal for 3-D modeling. MIKE 3 can be applied to perform analysis in the areas such as: hydrodynamics; sediments; water quality and ecology; and performance and parallel computing.

MIKE 3 is the ideal software for:

- Lake Hydrodynamics and ecology.
- Coastal and marine restoration projects.
- Coastal and oceanographic circulation studies including fine sediment dynamics.
- Optimization of coastal, thermal or wastewater disposal outlets.
- Analysis and optimization of cooling water recirculation and desalination.
- Environmental impact assessment of marine infrastructures.
- Assessment of hydrographic conditions for design, construction and operation of structures and plants in stratified waters.

- Ecological modeling including optimization of aquaculture systems.

The model solves the 3-D incompressible Reynolds averaged Navier-Stokes equations. Both the full 3-D Navier-Stokes equations (non-hydrostatic) and the 3-D shallow water equations (hydrostatic) can be applied. Thus, the model solves continuity, momentum, and temperature, salinity, and density equations and is closed by a turbulent closure scheme. The spatial discretization of the governing equations is performed using a cell-centered finite volume method on triangular, quadrangular or mixed element domains, employing a shock-capturing Riemann solver to ensure robust and stable simulation of flows.

MIKE 3 is modular. Users can select the modules they need. MIKE 3 includes a wide range of modules, allowing users to create their own tailored modeling framework for their studies.

- PP - Preprocessing and Post-processing: This module offers an integrated work environment which provides convenient and compatible routines to ease the task of data input, analysis and presentation of simulation results similar to that of MIKE 21.
- HD – Hydrodynamics: This module simulates the water level variations and free surface flows in response to a variety of forcing functions. It includes a wide range of hydraulic phenomena in the simulations. The Flexible Mesh engine, which uses a depth and surface adaptive vertical grid, is particularly suitable in areas with a high tidal range.
- AD - Advection-Dispersion: This simulates the transport, dispersion and decay of dissolved or suspended substances. It is typically used in cooling water and sewage outfall studies.
- ABM LAB - Agent Based Modeling: This is a flexible numerical laboratory used to define agents, their behavior and states.
- UAS – Underwater Acoustic Simulator: This module offers modeling of the propagation of underwater noise from a variety of man-made activities at sea. It is the ideal tool for managing noise impacts.

The Hydrodynamic module features include: flood and drying; bed resistance; density variations; transport of salinity and temperature; turbulence modeling including buoyancy effects; wind friction and/or velocity boundaries; isolated sources and sinks, connected source/sink pairs; pier resistance; heat exchange with atmosphere including evaporation/precipitation; particle tracking; and discharge calculations etc.

MIKE 3 includes the following modules specifically for sediment transport and water quality modeling.

- ST - Sand Transport: The advanced sand transport model in MIKE 21 has been ported to MIKE 3 and dynamically coupled to the 3-D hydrodynamic flow model. MIKE 3 ST module includes two options for extracting 2-D information from the 3-D flow: mean and derivation or bed shear stress. This extends the use further into, for example, river morphology and to areas with current circulation such as confined bays.
- MT - Mud Transport: This is a combined multi-fraction and multi-layer model that describes erosion, transport and deposition of mud (cohesive sediments). A dredging module has been added to the features of the MT module, allowing dynamic simulation of all stages of the dredging process.
- PT - Particle Tracking: This module simulates transport and fate of dissolved and suspended substances. It is, for example, used for risk analyses, accidental spillage and monitoring of dredging works.

- WFM – Waves: The state-of-the-art flexible mesh tool for simulating fully non-linear and fully dispersive 3-D wave kinematics with no depth restrictions in the model domain. Featuring excellent flood & dry capabilities, the tool handles run-up and overtopping events in coastal flooding studies exceptionally well.
- Boundary Conditions Generator: Create high quality hydrodynamics boundary conditions automatically.

CE-QUAL-W2- Refer Figure D-102

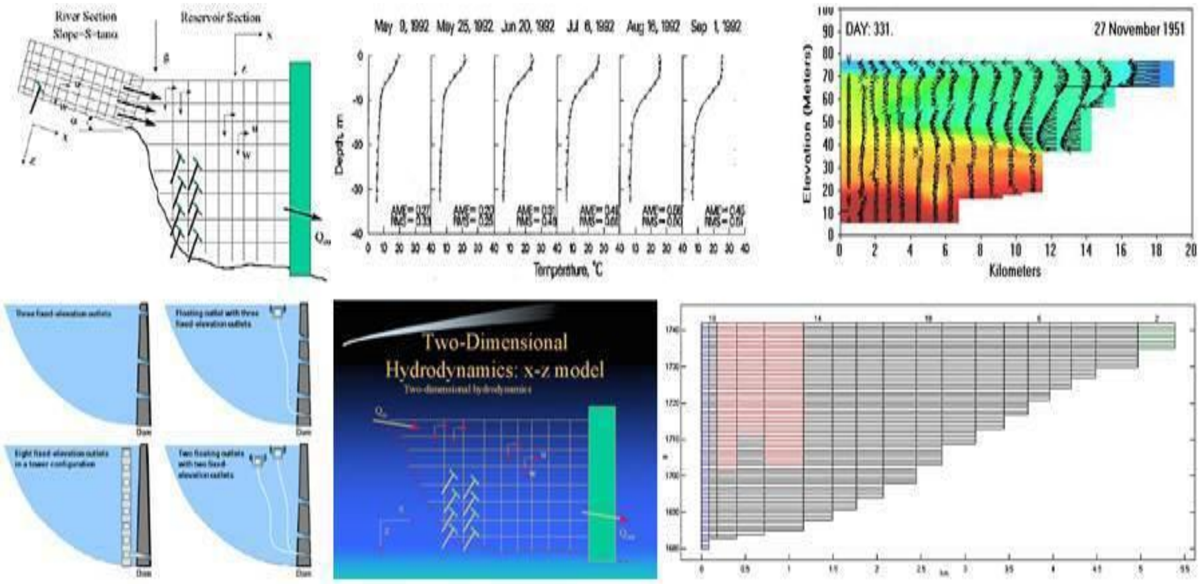


Figure D- 102 CE-QUAL-W2

CE-QUAL-W2 is a two-dimensional, longitudinal/vertical, laterally-averaged hydrodynamic and water quality simulation model. The model allows for application to streams, rivers, lakes, reservoirs, and estuaries with variable grid spacing, time-variable boundary conditions, and multiple inflows and outflows from point/nonpoint sources and precipitation. Since the model assumes lateral homogeneity, it is best suited for relatively long and narrow water-bodies exhibiting longitudinal and vertical water quality gradients. The model predicts water surface elevations, velocities, and temperatures.

Temperature is included in the hydrodynamic calculations because of its effect on water density. The model can be applied to estuaries, rivers, or portions of a water-body by specifying upstream or downstream head boundary conditions. The branching algorithm allows application to geometrically complex water-bodies such as dendritic reservoirs or estuaries. The water surface elevation is solved implicitly which eliminates the surface gravity wave restriction on the time-step. This permits larger time-steps during a simulation resulting in decreased computational time. As a result, the model can easily simulate long-term water quality responses. In CE-QUAL-W2, the laterally-averaged three-dimensional continuity and momentum (conservation of the fluid mass and conservation of momentum, respectively) equations that govern reservoir hydrodynamics are resolved numerically using finite difference methods. Furthermore, the model solves the two-dimensional advection-diffusion equation for water temperature and other water quality parameters such as suspended solids, nutrients, biological oxygen demand and algal dynamics. In order to control the model's numerical stability, the model allows for the use of dynamic time steps where regions with frequent flow fluctuations can temporarily use smaller time steps when necessary. The CE-QUAL-W2 model computes 28 state variables, 23 derived water quality variables and 73 water quality fluxes.

The two major components of the model include hydrodynamics and water quality kinetics. Both of these components are coupled, i.e. the hydrodynamic output is used to drive the water quality at every time-step. The hydrodynamic portion of the model predicts water surface elevations, velocities, and temperature. The water quality portion can simulate more than 20 constituents including DO, nutrients, phytoplankton interactions, and pH. A dynamic shading algorithm is incorporated to represent topographic and vegetative cover effects on solar radiation. This model has been extensively tested, documented, and applied to environmental studies worldwide by universities, governmental agencies, and environmental consulting firms. The water quality algorithm is modular allowing constituents to be easily added as additional subroutines. The CE-QUAL-W2 model is a data intensive application. The data required for an application include bathymetric data, meteorological data (air temperature, dew point temperature, wind speed, wind direction, cloud cover, solar radiation, and precipitation), inflow and outflow volumes, inflow temperatures, evaporation, water quality constituent concentrations, and hydraulic and kinetic parameters. The availability and quality of these data directly affect model accuracy and limit usefulness.

CE-QUAL-W2 has been under continuous development since 1975. The original model was known as LARM (Laterally Averaged Reservoir Model) developed by Edinger and Buchak in 1975. The first LARM application was on a reservoir with no branches. Subsequent modifications to allow for multiple branches and estuarine boundary conditions resulted in the code known as GLVHT (Generalized Longitudinal-Vertical Hydrodynamics and Transport Model). Addition of the water quality algorithms by the Water Quality Modeling Group at the USACE Waterways Experiment Station (WES) resulted in CE-QUAL-W2 Version 1.0 (Environmental and Hydraulic Laboratories 1986). CE-QUAL-W2 Version 2.0 is a result of major modifications to the code to improve the mathematical description of the prototype and increase computational accuracy and efficiency.

CE-QUAL-W2 is developed primarily for use in simulating reservoir dynamics and has many features required for reservoir simulations that are not found in standard estuarine models. WES has been continuing the effort of improvement and updating of CE-QUAL-W2 through the releases of newer versions.

Key capabilities of CE-QUAL-W2 are provided as follows:

- Ability to model multiple inflows, outflows, and water-bodies including multiple reservoirs, steeply sloping riverine sections between reservoirs, and estuaries.
- Varying vertical grids between water-bodies.
- The effect of lateral inflows from tributaries on the vertical eddy viscosity with inclusion of momentum transfer between branches.
- Turbulence closure models for each water-body using eddy-viscosity mixing length models.
- Turbulent kinetic energy-turbulent dissipation turbulence closure model.
- Vertical turbulence and reaeration algorithms for rivers.
- Reaeration formulae based on the riverine or reservoir/lake.
- An implicit solution for the effects of vertical eddy viscosity.
- Use Chezy or Manning's friction-factor.
- Time-weighted vertical advection and fully implicit vertical diffusion.

- The computation of the drag coefficient for low wind speeds.
- A selective withdrawal algorithm that calculates a withdrawal zone based on outflow, outlet geometry, and upstream density gradients.
- Automatic selection of a withdrawal port algorithm that will select the elevation of the withdrawal necessary to meet temperature targets including splitting flows between outlets to reach a target temperature.
- An algorithm to estimate suspended solids resuspension as a result of wind-wave action.
- Sediment/water heat exchange and a user-defined settling velocity.
- Numerical algorithms for pipe, weir, and pump flow.
- A dynamic pipe algorithm allowing a pipe to be turned ON or OFF over time, as if a gate was closed.
- A dynamic pump algorithm that allows the model user to set dynamic parameters for the water level control over time.
- An ice-cover algorithm.
- Internal calculation of equilibrium temperatures and coefficients of surface heat exchange or a term-by-term accounting of surface heat exchange.
- An algorithm that adjusts the time-step to ensure hydrodynamic stability requirements.
- Generalized time-varying data input subroutine.
- Variable layer heights and segment lengths and surface layer extending through multiple layers.
- Arbitrary constituents defined by a decay rate, settling rate, and temperature rate multiplier.
- A graphical pre/postprocessor with option for output is in the TECPLOT format.
- A restart provision.

The governing equations for hydrodynamics and transport are laterally and layer averaged. Lateral averaging assumes lateral variations in velocities, temperatures, and constituents are negligible. This assumption may be inappropriate for large water-bodies exhibiting significant lateral variations in water quality. Whether this assumption is met is often a judgment call on the user and depends in large part on the questions being addressed. Eddy coefficients are used to model turbulence. The equations are written in the conservative form using the Bossiness and hydrostatic approximations. Since vertical momentum is not included, the model may give inaccurate results where there is significant vertical acceleration.

The model can place significant computational and storage burdens on a computer when making long-term simulations. Applications to dynamic river systems can take considerably longer than reservoirs because of much smaller time-steps needed for river numerical stability. The application of CE-QUAL-W2 to perform a hydrodynamics/water quality study requires knowledge in the following areas: Hydrodynamics; Numerical methods; Aquatic biology; Aquatic chemistry; Computers and FORTRAN coding; Statistics; and Data assembly and reconstruction. Water quality modeling is in many ways an art requiring not only knowledge in these areas but also experience in their integration.

CE-QUAL-R1- Refer Figure D-103

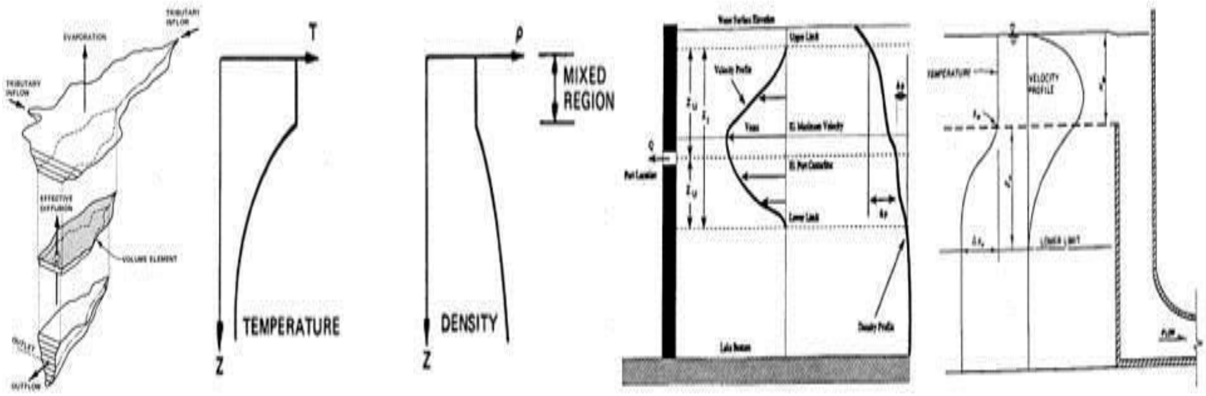


Figure D- 103 CE-QUAL-R1

CE-QUAL-R1 is a mathematical simulation model of water quality that describes the vertical distribution of thermal energy and biological and chemical materials in a reservoir through time. It is used to study pre-impoundment and post-impoundment water quality problems and the effects of reservoir management operations on water quality. The model also addresses problems of water quality associated with reservoir eutrophication with possible anaerobic conditions. CE-QUAL-R1 is spatially one dimensional and horizontally averaged; temperature and concentration gradients are computed only in the vertical direction. The reservoir is conceptualized as a vertical sequence of horizontal layers where thermal energy and materials are uniformly distributed in each layer. CE-QUAL-R1 simulates the dynamics of 27 water quality variables, calculating both vertical in-pool and downstream release magnitudes. In addition, 11 other variables, which represent materials in the sediments, are modeled.

The model requires an extensive data base including initial conditions, geometric and physical coefficients, biological and chemical reaction rates, and time sequences of hydro meteorological and inflowing water quality concentrations. A substantial effort by a multi-disciplinary staff is necessary to acquire data; create and debug the data base; add site-specific factors to the computer code as necessary; and compute, plot, and analyzes results. Experience suggests that staff personnel need several months to accomplish these tasks. Typically, 40 percent of the time will be spent on data preparation, 10 percent on computation, and 50 percent on analysis of output.

The mathematical structure of the model is based on horizontal layers whose thicknesses depend on the balance of inflowing and outflowing waters. Variable layer thicknesses permit accurate mass balancing during periods of large inflow and outflow. The distribution of inflowing waters among the horizontal layers is based on density differences. Simulations of surface flows, interflows, and underflows are possible. Similarly, outflowing waters are withdrawn from layers after considering layer densities, discharge rates, and outlet configuration. Reservoir outflows may take place according to a specified schedule of port releases. Alternately, specification of total release and desired release temperatures can be made. In this case, the model will select port flows. In addition, both continuous (normal) and scheduled operations can be simulated. Continuous operation refers to normally uninterrupted port or weir outflows. Scheduled operation refers to fluctuating generation outflows or pump-back inflows.

Vertical transport of thermal energy and materials occurs through entrainment and turbulent diffusion. Entrainment is a transport process that sharpens gradients and determines the depth of the upper mixed region and the onset of stratification. It is calculated from the turbulent kinetic energy influx generated by wind shear and connective mixing. Turbulent diffusion is a transport process that reduces gradients and is calculated using a turbulent diffusion coefficient that is dependent on wind speed, inflow and outflow magnitudes, and density stratification. The model simulates interactions of physical factors (such as flow and temperature), chemical factors (such as nutrients), and biological assemblages in both aerobic and anaerobic environments. The model can perform stochastic simulations using Monte Carlo methods. Statistical data describing biological and chemical coefficients are used to provide probabilistic estimates of key output variables. The thermal analysis portion of CE-QUAL-R1 is provided as an independent model (CE-THERM-R1) to simplify simulation of water budgets and temperature profiles. CE-THERM-R1 includes the variables of temperature, suspended solids, and total dissolved solids. Algorithms representing physical processes are the same as in CE-QUAL-R1.

CE-QUAL-R1is a direct descendant of the reservoir portion of an earlier model called "Water Quality for River-Reservoir Systems" (WQRRS). WQRRS was assembled for the Hydrologic Engineering Center of the Corps of Engineers by Resource Management Associates, Inc. (1978).

WQRRS- Refer Figure D-104

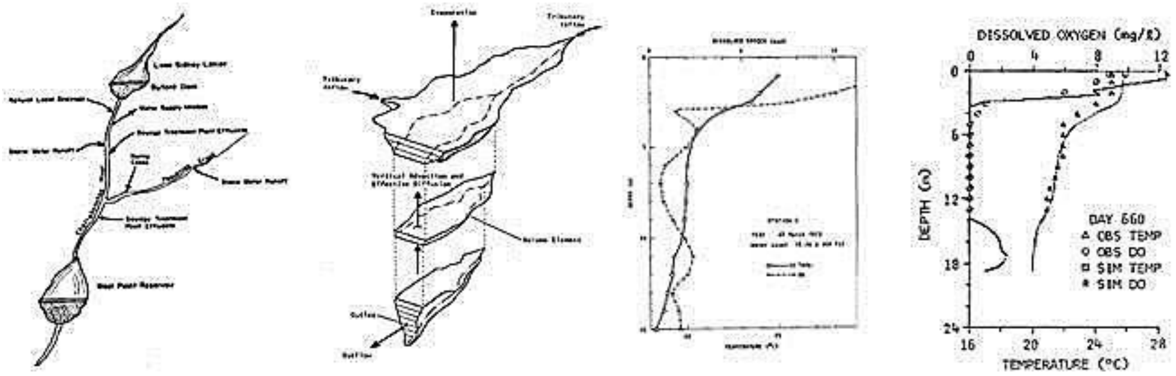


Figure D- 104 WQRRS

The WQRRS is capable of representing dynamic, one-dimensional flow and water quality in rivers and reservoirs. WQRRS model consists of three separate but integral modules: the reservoir module, the stream hydraulic module, and the stream quality module. The reservoir and stream hydraulics modules are stand-alone programs and may be independently executed, analyzed, and interpreted. The stream quality module, however, has no hydraulic computation capability and requires a hydraulic data file which is generated by the stream hydraulics module. The three computer programs may also be integrated for a complete river basin water quality analysis through automatic storage of results for input to downstream simulations. The subsequent analysis may be a part of the same simulation or an entirely separate model execution. Input/output compatibility for downstream analysis is consistent among modules. Many subroutines are similar, if not identical, among the reservoir and stream modules. The methodology in the reservoir section of the program is applicable to aerobic impoundments that can be represented as one-dimensional systems in which the isotherms, or indeed the contours of any parameter, are horizontal. This approximation is generally satisfactory in small to moderately large lakes or reservoirs with long residence times. The approximation may be less satisfactory for shallow impoundments or those that have a rapid flow-through time. Although systems that have a rapid flow-through time are often fully mixed and can often be treated as slowly moving streams using the stream section of the model.

DYRESM- Refer Figure D-105

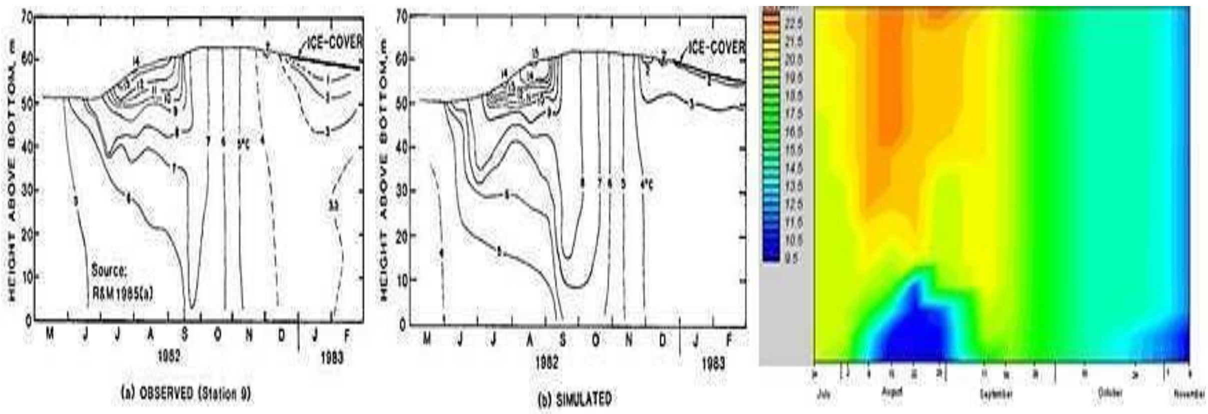


Figure D- 105 DYRESM

DYRESM (Dynamics Reservoir Simulation Model) is based on an assumption of one dimensionality that the hydrodynamic variations in a vertical direction are much greater than horizontal variations, and thus there is no lateral or longitudinal variation across a water body. The model predicts the vertical distribution of temperature, salinity, and density in lakes and reservoirs that are predominantly one-dimensional and is suitable for simulating the hydromechanics of small to medium-sized impoundments. The horizontal transport in stratified lakes and reservoirs are typically several orders of magnitude greater than the vertical transport rates since most lakes are laterally homogenous but vertically stratified. The model is formulated based on the parameterization of the principal physical processes responsible for the mixing of heat and other water quality components. In the model, mixing is simulated using the principles of conservation of mass and momentum. Calculations are applied to each layer, starting at the surface, determining layer by layer whether there is enough free energy for two interfacing layers to mix. It provides daily predictions of the temperature and salinity variations with depth and the temperature and salinity of the off-take supply. Within each daily step, a variable time step (as short as 1/4 hours) is used for modeling the mixing processes. The reservoir is modeled by a set of horizontal layers of variable thickness which alter their vertical location, volume, temperature and salinity as the kinematics require. No convection occurs across layer boundaries, as mixing is accomplished by layer amalgamation. Inflow and withdrawal are modeled by layer thickening and depletion, respectively. The vertical transport above inflow or withdrawal is modeled by the vertical shift of layers. The horizontally averaged profiles of suspended sediment are changed in the model by three processes; namely by vertical mixing, inflows and outflows, and settling.

DYRESM is capable of describing the dynamics of lakes and reservoirs given that the time scales of extreme events such as floods and storms are small, and can help predict the seasonal and annual variability of lakes and reservoirs. Furthermore, the model is capable of testing the sensitivity of a lake to long term changes in environmental factors. The model requires substantial and accurate input data files, including: meteorological data, lake morphology, initial water profiles, inflows and outflows. DYRESM is not computationally demanding and can easily simulate tens of years' worth of data without any problems. The model has been well tested based on several reservoir observations and has been applied to many lakes, worldwide. DYRESM was developed by the Center for Water Research at the University of Western Australia and has been updated and enhanced regularly as a result of continuous effort by many researchers.

FLOW-3D- Refer Figure D-106

Flow-3D is an advanced CFD numerical modeling software which is capable of solving a wide range of hydraulic and fluid flow problems. Areas of software application include the hydroelectric and dam and irrigation projects; and aerospace industry etc. A good selection of different options across the entire Flow-3D graphical user interface allows the software to be applicable to such a wide variety of situations. Flow-3D allows either one or two fluid flow, with or without a free surface, and a multitude of available physics options to suit the specific application. Various meshing and geometry options are available including multi-block grids and the ability to draw simple objects in the software or import different forms of more complex geometry or topographic files. A large selection of boundary conditions is also available to properly model each specific application. Another benefit of Flow-3D is the ability to select from several different implicit and explicit numerical solver options. All of these model set-up parameters can easily be specified by either encoding selections in the text editor or by making radio-button selections in the graphical user interface. FLOW-3D uses special numerical methods to track the location of surfaces and applies the proper dynamic boundary conditions at those surfaces. Free surfaces are modeled with the Volume of Fluid (VOF) technique that was first developed by a group of scientists, including Flow Science's founder, Dr. C. W. Hirt, at the Los Alamos National Laboratory.

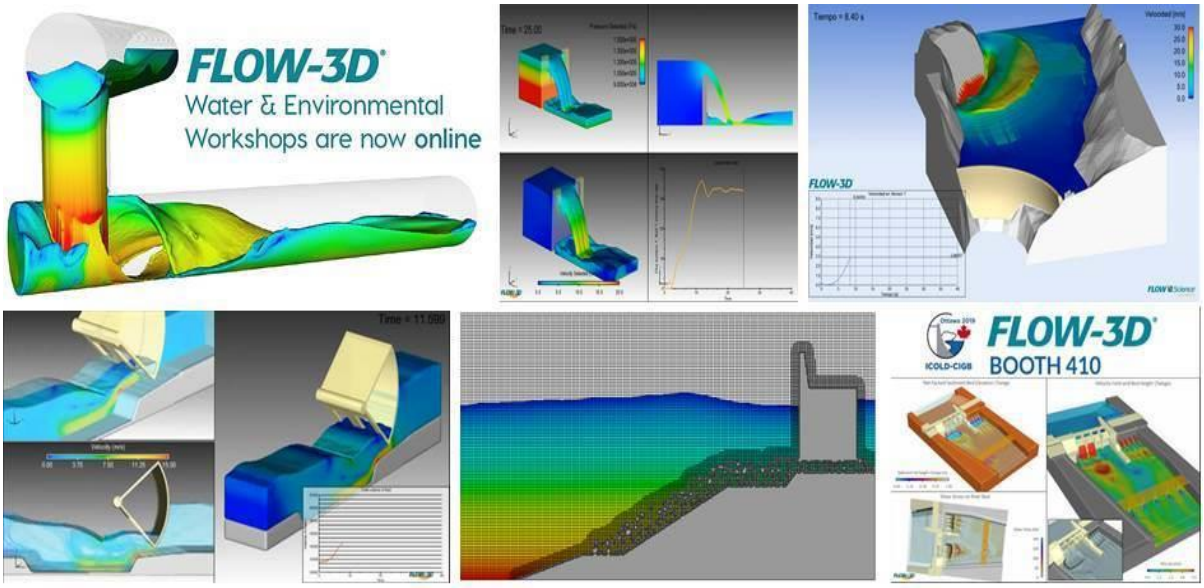


Figure D- 106 FLOW-3D

FLOW-3D incorporates its own VOF methodology called TruVOF which includes major improvements beyond the original VOF method to increase the accuracy of boundary conditions and the tracking of interfaces. FLOW-3D also incorporates a special technique, known as the FAVOR™ (Fractional Area Volume Obstacle Representation) method, which is used to define general geometric regions within the rectangular grid. The concept behind FAVOR™ is that numerical algorithms are based on information consisting of only one pressure, one velocity, one temperature, etc., for each control volume, so it would be inconsistent to use much more information to define the geometry. Thus, the FAVOR™ technique retains the simplicity of rectangular elements while representing complex geometric shapes at a level consistent with the use of averaged flow quantities within each volume element. FLOW-3D has been selected by many project engineers to carry out complex hydraulic analysis primarily for its ability to accurately model free surface flow, which is essential for modeling the open-channel flow behavior that is commonly found in spillway flows. It utilizes the true volume of fluid (TruVOF) method for computing free surface motion and the fractional area/volume obstacle representation (FAVOR) technique to model complex geometric regions. The TruVOF method tracks the sharp interface accurately and does not require computation of the dynamics in the void or air regions. If air-water interaction is deemed not important, the ―single fluid‖ approach allows faster run-time.

The ability to model wall roughness, air entrainment and cavitation are also important considerations in selecting the code. The use of structural Cartesian grids or meshes meant the meshing process could be done efficiently. The mesh is overlaid on the imported non-flow geometry and the FAVOR technique is used to determine the void or flow region within each cell. With finer grid spacing, the higher resolution of the non-flow region (obstacle) is achieved. The use of multi-block grids enables larger domains to be modeled and the use of nested mesh blocks enables more flow details to be captured in regions of interest. The code assumes the ―law of the wall‖ to mimic the flow behavior close to obstacles. The code solves the momentum and mass-transport equations in a transient, time stepping manner to reach a dynamic ―steady state.‖ The CFD code, FLOW-3D, was developed by Flow Science, Inc. Flow Science has also released a HPC (High Performance Computing)-enabled version of FLOW-3D. This HPC version can be run on in-house clusters or on the FLOW-3D CLOUD software as a service platform as described below.

HPC-Enabled FLOW-3D v12.0

The FLOW-3D version 12.0 is HPC-enabled version released by Flow Science. This HPC version can be run on in-house clusters or on the FLOW-3D CLOUD software as a service platform, which provides high performance computing (HPC) as well as the lowest cost entry-point to use FLOW-3D.

FLOW-3D can be run on a single core or 1000s of CPU-cores, FLOW-3D is engineered to take full advantage of the ongoing advancements in hardware. The high performance computing and FLOW-3D CLOUD with an emphasis on deploying hardware and software resources on demand, such as performance benchmarks for understanding scaling and speed-up on the cloud are the key features of FLOW-3D v12.0. This version provides users the design and functionality of the graphical user interface, which simplifies model setup and improves user workflows. A state-of-the-art Immersed Boundary Method brings greater accuracy to obtain solutions. Features include the 2-Fluid 2-Temperature Model, the Steady State Accelerator, which allows users to model their free surface flows even faster. The HPC-enabled version of FLOW-3D v12.0 allows users to access these advanced simulation options at an accelerated pace. From running design variations simultaneously to solving fine-resolution, large, and highly-complex design scenarios that take weeks to run on a high-end workstation, the HPC-enabled version produces the answer faster on user‘s in-house cluster or on the cloud platform of Flow Science.

ANSYS FLUENT- Refer Figure D-107

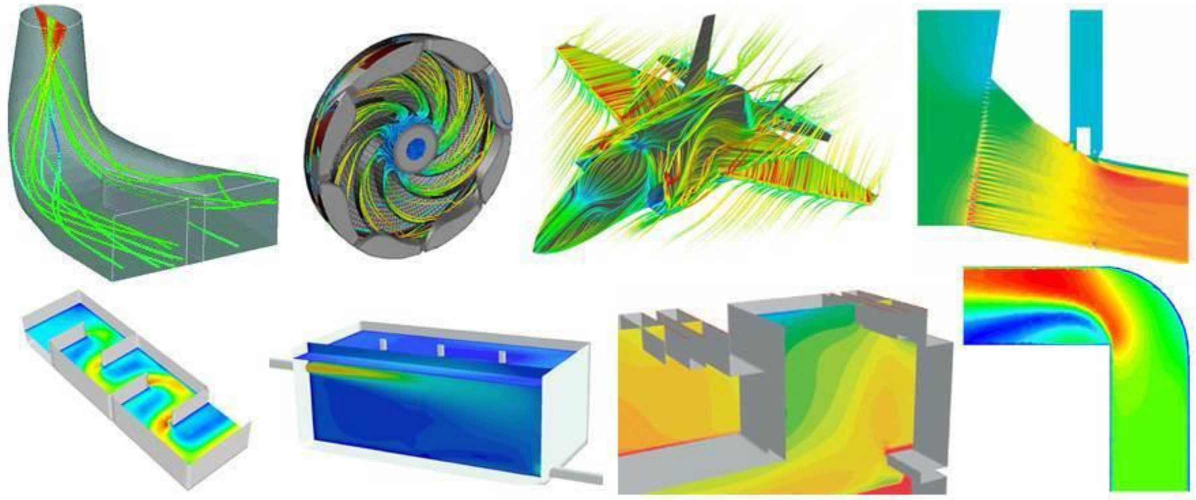


Figure D- 107 ANSYS FLUENT

The ANSYS FLUENT CFD software suite provides comprehensive modeling capabilities for a wide range of incompressible and compressible, laminar and turbulent fluid flow problems. In ANSYS FLUENT, a broad range of mathematical models for transport phenomena (such as heat transfer and chemical reactions) is combined with the ability to model complex geometries. Examples of applications include external aerodynamics; flow through compressors, pumps, fans; multiphase flows; laminar non-Newtonian flows in process equipment; conjugate heat transfer in turbomachinery and automotive engine components, etc. For dam projects, a very useful group of models in ANSYS FLUENT is the set of free surface and multiphase flow models. These can be used for analysis of gas-liquid, gas-solid, liquid-solid, and gas-liquid-solid flows. ANSYS FLUENT provides the volume-of-fluid (VOF), mixture, and Eulerian models, as well as the discrete phase model (DPM). The DPM performs Lagrangian trajectory calculations for dispersed phases (particles, droplets, or bubbles), including coupling with the continuous phase. Examples of multiphase flows include channel flows, sprays, sedimentation, separation, and cavitation. To permit modeling of fluid flow and related transport phenomena in industrial equipment and processes, various useful features are provided. These include porous media, lumped parameter (fan and heat exchanger), stream wise-periodic flow and heat transfer, swirl, and moving reference frame models. The moving reference frame family of models includes the ability to model single or multiple reference frames. A time-accurate sliding mesh method, useful for modeling multiple stages in turbo-machinery applications, for example, is also provided, along with the mixing plane model for computing time-averaged flow fields.

Robust and accurate turbulence models are a vital component of the ANSYS FLUENT suite of models. The turbulence models provided have a broad range of applicability, and they include the effects of other physical phenomena, such as buoyancy and compressibility. Particular care has been devoted to addressing issues of near-wall accuracy via the use of extended wall functions and zonal models. Various modes of heat transfer can be modeled, including natural, forced, and mixed convection with or without conjugate heat transfer, porous media, etc. The set of radiation models and related sub-models for modeling participating media are general and can take into account the complications of combustion.

In 1996, FLUENT acquired FIDAP from Fluid Dynamic International (FDI) and FLUENT was later in 2006 purchased by ANSYS and became ANSYS FLUENT. ANSYS also acquired CFX from UK Atomic Energy Authority (AEA) in 2003 after AEA purchased TASCflow from Advanced Scientific Computing (ASC-Canada) in 2003. FIDAP and TASCflow are briefly described as follows.

FIDAP software can be used to perform a wide range of fluid flow analyses. FIDAP uses the finite element method, which can treat arbitrarily complex flow domains and boundary conditions. Unstructured grids can be designed that allow areas of interest to be studies without the need for excessive grid points throughout the entire flow domain. Some heat transfer applications include: (1) temperature variations caused by heat sources and sinks, or convective and radiative heat flux across boundaries, (2) heat generation due to viscous dissipation, (3) conjugate heat transfer, (4) effuse gray-body radiation, (5) solidification and melting, with latent heat release and mass transfer across phase interfaces, and (6) heat exchanger.

TASCflow software package provides numerical modeling of three dimensional incompressible or compressible, laminar or turbulent, subsonic, transonic and supersonic viscous flows. TASCflow uses finite element based conservative finite volume methods for analyzing fluid flow in stationary and rotating components of rotating machinery (pumps, turbines, propellers, etc.). The program models flow within spiral casing, stay vanes, wicket gates, and draft tubes etc. and may be applied to general fluid flow, heat transfer and combusting and reacting flow analysis. Methods of solution include advanced discretization schemes, fully conservative grid embedding, and coupled algebraic multi-grid methods. This software package is a complete Computational Fluid Dynamics (CFD) model that includes pre- and post-processing software and a CADD-based grid generator which allows for importation of grid information from CADD files. A multi-grid solver accounts for coupling between momentum equations caused by Coriolis forces, centrifugal forces, and swirling flows. Erosion in particle-laden flows can be modeled with an available Lagrangian tracking module.

FENFLOSS- Refer Figure D-108

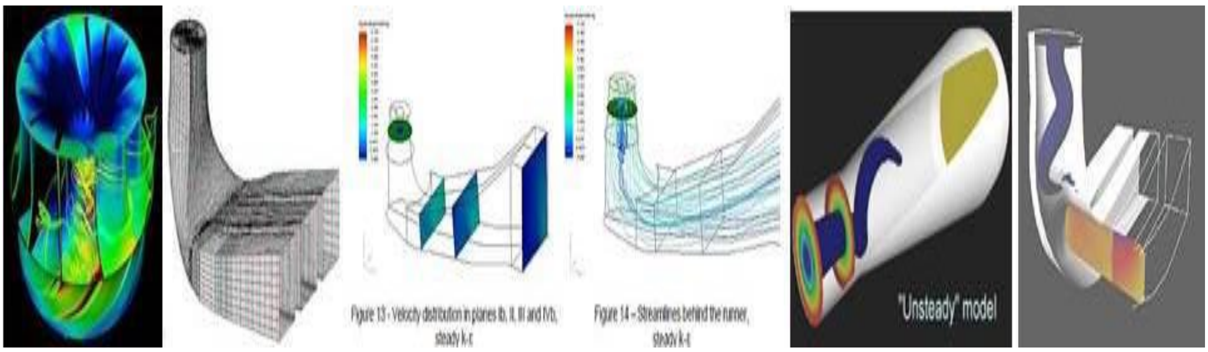


Figure D- 108 FENFLOSS

The program FENFLOSS is developed at the Institute of Fluid Mechanics and Hydraulic Machinery, University of Stuttgart. It is based on the Finite Element Method. For spatial domain discretization 8-node hexahedral elements are used and time discretization involves a three-level fully implicit finite difference approximation of 2nd order. For the velocity components and the turbulence quantities a trilinear approximation is applied and the pressure is assumed to be constant within each element. For advection dominated flow a Petrov-Galerkin formulation of 2nd order with skewed upwind orientated weighting function is used. The various turbulence models such as the standard k-ε model are implemented into FENFLOSS. Using supercomputers, FENFLOSS can perform simulations with highly accurate numerical schemes and sophisticated turbulence models.

FENFLOSS are often applied to perform hydraulic analysis of components of a hydropower plant such as the flow through a turbine and draft tube for stability and reliability in order that hydropower can play an essential role for integrating volatile renewable energy source into the electrical net. Flow in a draft tube is characterized as very intricate turbulent flow followed with appearance of different flow phenomena, e.g. unsteadiness, flow separation, swirling flow etc. Its analysis and simulation are complicated and time consuming requiring high computational power. Due to the strong increase of fluctuating renewable energies, a great amount of regulating power is necessary in the net, which is mainly available from hydropower. As a consequence, the hydraulic turbines are very often operated in extreme off-design conditions. The occurrence of vortex rope of the flow in a draft tube often resulted in undesirable pressure pulsations and should be minimized. FENFLOSS can be used to analyze complex flow structures and frequent flow instabilities occur in the turbine unit, which can cause severe dangerous oscillations in the entire hydroelectric plant [67, 70, 118, 119, and 120].

STAR-CCM+- Refer Figure D-109

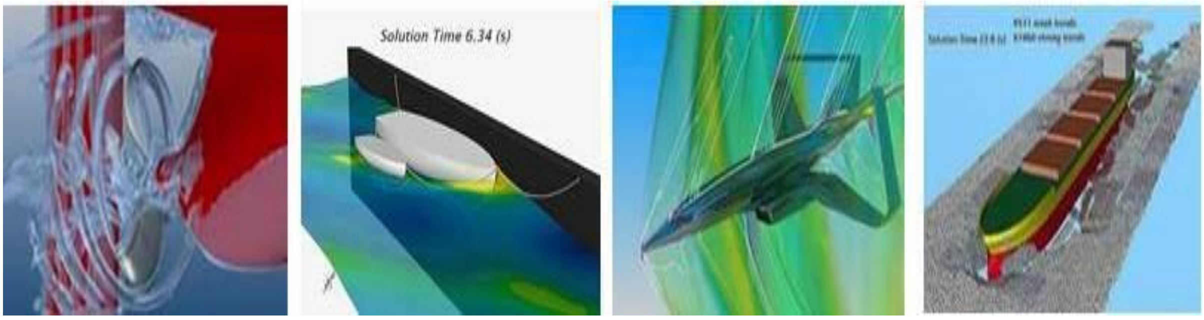


Figure D- 109 STAR-CCM+

Star-CCM+ is a CFD solver which is developed and distributed by CD-adapco. It was initially designed to understand the flow characteristics around obstacles and inside fluidic system. The software has emerged as a tool for coupled multiphysics simulations. STAR-CCM+ offers double precision and so called mixed precision versions. The mixed precision version uses mainly single precision arithmetic, with a few subroutines/variables that are in double precision accuracy.

CD-adapco (Computational Dynamics-Analysis & Design Application Company Ltd) was a multinational computer software company that authored and distributed application software used for computer-aided engineering. In April 2016, CD-adapco was acquired by Siemens.

PHOENICS- Refer Figure D-110

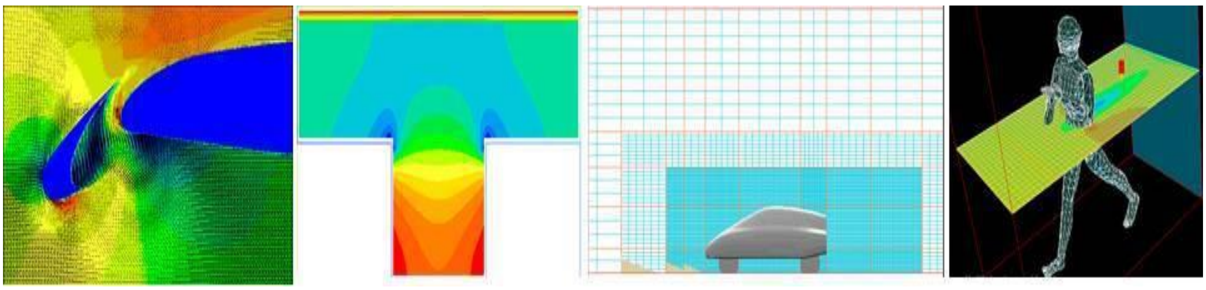


Figure D- 110 PHOENICS

PHOENICS is developed and maintained by CHAM. It is a CFD program for simulating scenarios involving fluid flow, heat or mass transfer, chemical reactions and combustion for a wide range of applications. PHOENICS is a general-purpose software package which uses the techniques of CFD (i.e. Computational Fluid Dynamics) to predict quantitatively: how fluids (air, water, steam, oil, blood, etc) flow in and around: lakes, river and oceans, engines, process equipment, buildings, human beings, and so on; what are the associated changes of temperature and of chemical and physical composition. Its name is an acronym for Parabolic Hyperbolic or Elliptic Numerical Integration Code Series, wherein "parabolic", "hyperbolic" and "elliptic" are the words which mathematicians use to distinguish the underlying equations. PHOENICS is employed primarily by: engineers for the design of aircraft and other vehicles, and of equipment which produces power or which processes materials; architects for the design of buildings; environmental specialists for the prediction, and if possible control, of environmental impact and hazards; scientists for interpreting their experimental observations; and institutions for the study of fluid dynamics, heat transfer, combustion and related disciplines. PHOENICS has been continuously marketed, used and developed since 1981.

Open TELEMAC-MASCARET- Refer Figure D-111

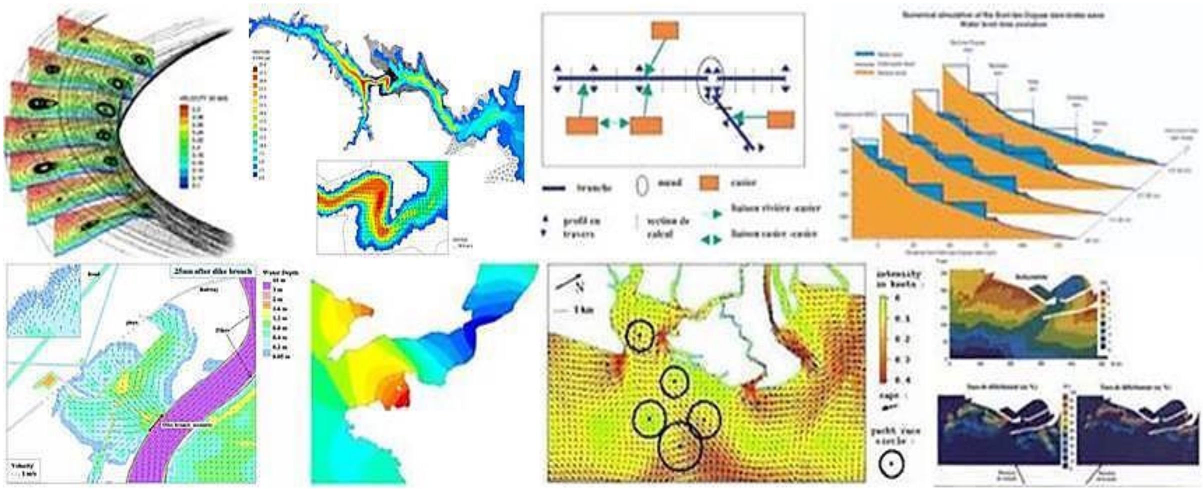


Figure D- 111 Open TELEMAC-MASCARET

TELEMAC-MASCARET is an integrated suite of solvers for use in the field of free-surface flow. It is managed by a consortium of core organizations: Artelia (formerly Sogreah, France), BundesAnstaltfürWasserbau (BAW, Germany), Centre d'Etudes Techniques Maritimes et Fluvial (CETMEF, France), Daresbury Laboratory (United Kingdom), Electricité de France R&D (EDF, France), and HR

Wallingford (United Kingdom). The system was developed by the Laboratoire National d'Hydraulique, a department of Electricité de France's Research and Development Division. It is now available as open source, and is maintained by the Open Telemac-Mascaret Consortium. The TELEMAC consortium maintains a collaboration website to help the community to make the most of the open-source TELEMAC-MASCARET.

The TELEMAC-MASCARET modeling system.

The system is an integrated modeling tool for use in the field of free-surface flows. Having been used in the context of many studies throughout the world, it has become one of the major standards in its field. The various simulation modules use advanced algorithms based on the finite-element method. Space is discretized in the form of an unstructured grid of triangular elements, which means that it can be refined particularly in areas of special interest. This avoids the need for systematic use of embedded models, as is the case with the finite-difference method. All the numerical algorithms are gathered into a single library that is shared by all the simulation modules. This makes for consistency throughout the TELEMAC-MASCARET system. The pre- and post-processing tools are powerful and user-friendly. The grid can be generated with the generator embedded in the TELEMAC-MASCARET system. TELEMAC-MASCARET has numerous applications in both river and maritime hydraulics. The various modules of the TELEMAC-MASCARET system run under: Windows (NT, XP, Vista, Win7); Linux (Ubuntu, Fedora, Redhat, OpenSUSE, Debian); UNIX and Supercomputers (Cray, Fujitsu, and IBM etc)

Structure of the TELEMAC-MASCARET system: The system consists of the following models:

Hydrodynamics

- ARTEMIS: Wave agitation in harbors.
- MASCARET: One-dimensional flows.
- TELEMAC-2D: Two-dimensional flows - Saint-Venant equations (including transport of a diluted tracer).
- TELEMAC-3D: Three-dimensional flows - Navier-Stokes equations (including transport of active or passive tracers).
- TOMAWAC: Wave propagation in the coastal zone.

Pre-/post-processors

- RUBENS: Graphical post-processor (now deprecated).
- MATISSE: Grid generation (now deprecated).
- STBTEL: Grid interface.
- POSTEL-3D: 2-D sections through the results of a 3-D simulation.

18. Micro Scale Physical Modeling for River Sedimentation Study

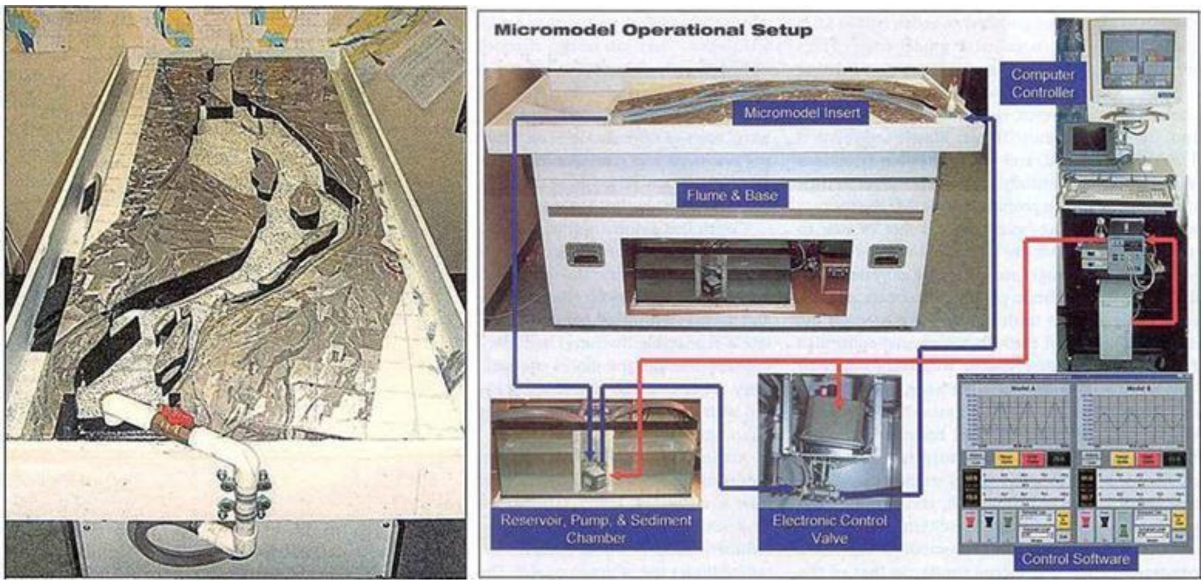


Figure D- 112 Micro model of the Mississippi Santa Fe Chute side channel and the operational setup.

A typical micro river model and the associated operational setup are shown in Figure D-112. When applied to the small-scale physical modeling of rivers and streams, micro modeling can help engineers and specialists solve a variety of flow and sedimentation problems [157]. Micro river models can also be used as an effective demonstration tool for engineers and managers to communicate with the public concerning river projects.

Micro modeling method is based on the principle that small streams display sediment transport tendencies similar to those of larger rivers. If the particular configuration of a river or stream is accurately constructed to a micro scale, the proper manipulation of certain variables in the micro model, including water flow rate, slope (i.e. floodplain tilt), and sediment, can produce similar sediment transport tendencies as those experienced in the actual prototype. These tendencies may be observed visually and may also be measured through a highly accurate procedure involving electronic digital micrometry [156]. As shown in Figure D-113, a typical micro river model [154] is significantly smaller than the small, tilting flume the St. Anthony Falls Hydraulic Laboratory used to conduct studies of river meandering [164].

A typical micro river model has the size roughly of a table top (Figure D-106). Based on a volumetric comparison, a typical river model built by the USACE Waterways Experiment Station (WES) is approximately 16,000,000 times smaller than the prototype, and a micro model is approximately 270,000,000 times smaller than the prototype [155]. It indicates that a micro model can be close to twenty times smaller than a WES river model.

Figure D- 113 A typical micro model (left) and the small flume model used by SAFHL for river meandering studies (right).

The Middle Mississippi as shown in Figure D-114 is a dynamic and fast-changing stretch of river. The Northern half contains locks and dams while the lower half is open-river. Each changing condition on the river creates the need for different solutions. Each solution, in its place, creates the opportunity for a diversity of habitats. The U.S. Army Corps of Engineers (USACE St. Louis District) has been developing new river structures to address sedimentation issues and to improve navigation and biological environment. The Middle Mississippi has been studied as an entire river system where different structures such as notched dikes, stepped-up dikes, revetments, off bank line revetments, chevron dikes, side channel improvements and bend way weirs (Figure D-117) were designed to fit specific locations on the river. Each structure was evaluated as to its ability to improve biological habitat and meet navigation goals, within the entire reach of the river. Before being installed in the river, many newly designed structures were model tested using either traditional large models or new micro modeling technology. Model testing evaluates various alterations and allows engineers to try nontraditional design approaches without the cost risks associated with field testing [154].

Figure D- 114 Photos of the Middle Mississippi.

Small micro scale physical mobile bed models have been used by USACE since 1994 for navigation design and environmental restoration on inland waterways of the United States. This relatively new modeling method is formally known as Hydraulic Sediment Response (HSR) modeling, which uses a small tabletop size physical model to simulate sediment conditions in rivers and streams. Its ability to replicate the observed bed response of the river has been established through continued advancements of the modeling technology. Comprehensive study has been carried out to compare micro model investigations to large-scale coal bed models used by USACE Engineer Research and Development Center (ERDC). The HSR micro modeling technology has been applied to study many reaches of the Missouri and Mississippi Rivers containing a large number of training structures. Experience accumulated through many years of applications of this modeling technique indicate that today‘s HSR micro models can replicate the bed response of the river with a high level of accuracy and within the observed natural variability of the river [158]. In 1997, this micro modeling method was granted a U.S. patent (Patent No. 5,653,592) to the U.S. Army of Corps of Engineers. The patent is titled —Method and Apparatus for Micro Modeling the Sediment Transport Characteristics of a River. The general layout of the patented micro model is shown in Figure D-115 [156].

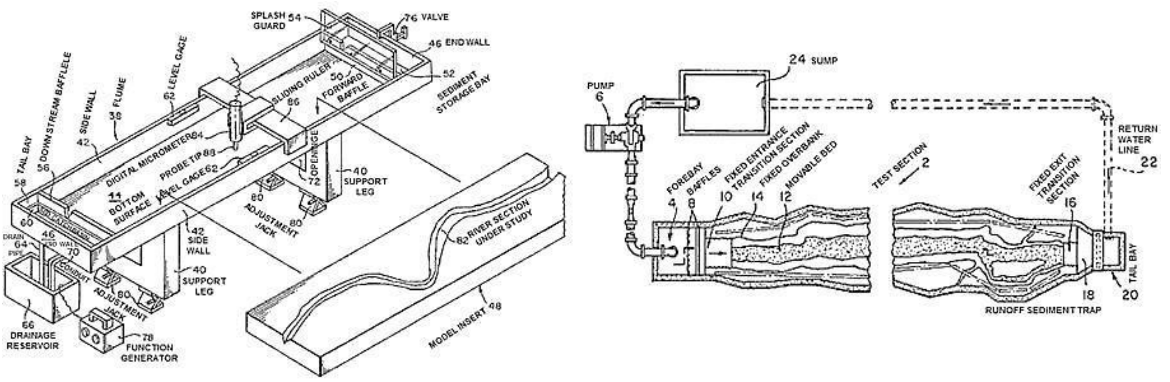


Figure D- 115 General design layout of the micro model (left) and a top view of the micro model.

Micro Modeling of the Mississippi River

Since the introduction of the micro models in 1994 by the St. Louis District of Corps of Engineers, the micro modeling technology has been used to address a variety of sedimentation problems and issues among them navigation design in the Mississippi River; restoration of side channels; bridge scour in rivers and streams; improvements to detrimental flow conditions at locks and dams; siltation at water supply intakes; and the effect of dredging. Micro modeling was developed on the basis of years of observation and analysis of Mississippi River data, including hydrographic surveys, velocity data, ice photos, and channel sweep and multi-swath bathymetry. Careful study of large physical sediment models used in Europe and the United States also played an important part in the development of the technology. The goal was to develop a practical applied engineering tool that could supply realistic answers to sedimentation problems in a cost effective and timely manner [154].

Figure D- 116 (a) Large Dogtooth Bend coal bed model (scale 1:400 horizontal, 1:100 vertical); (b) HSR Atchafalaya River micro model (scale 1:7200 horizontal, 1:1200 vertical); and (c) Conceptual comparisons of prototype and the traditional and micro models.

For comparison, a large traditional river model for the Dogtooth Bend and a micro river model for the Atchafalaya River are shown in Figures D-116 (a) and D-116 (b) respectively [158]. A conceptual comparison of prototype (for a 1000 feet reach) and the traditional and micro models is shown in Figure D-116c [155].

Environmental River Modeling

The Hydraulic Sediment Response (HSR) model is an excellent environmental engineering tool. HSR micro models have been used to address environmental issues on a number of Middle Mississippi side channels. The use of models has resulted in preservation and creation of habitat for fish and wildlife. These micro models have also been used to study methods to alleviate costly and harmful dredging, to modify river training structures for habitat creation, and to protect pristine environmental areas.

The use of the micro models enables the investigation team to address the complex sediment transport interaction problem between the side channels and the main navigation channel of the Mississippi River [159]. Navigation structures that work in harmony with the river have always been a priority. By developing a greater understanding of the need for habitat diversity through partnerships with river biologists, river engineers are able to design structures that afford an even greater harmony with the natural laws of the river without compromising navigation effectiveness. It‘s a situation in which everyone wins—man, nature and the river. The models have been used in conjunction with several biological impact studies to examine endangered species, including the Pallid Sturgeon and the Least Tern. The HSR micro models enable one to see and understand the interaction of large reaches of a river, and gain a keen understanding of how upstream changes can adversely or positively influence downstream conditions several miles away [154].

In the late nineteenth and early twentieth centuries, river engineers usually relied upon intuition, experimentation, and trial-and-error processes to design river training structures. Many of these projects caused long term effects to the river environment and wildlife habitat. Most modeling practices usually do not have the involvement of biologists and environmental scientists, etc. when designing structures and solving sediment related problems. Today, however, engineers are often tasked with correcting the negative environmental effects of those real life experiments and working closely with environmental scientists. There are many factors that contribute to a river‘s navigability as well as species diversity. The one factor that the engineers could impact was habitat. Habitats, such as fast water, slow water, quiet water and wetted-edge can be introduced through design modifications of navigation structure [154].

Developing designs for environment Friendly River training structures: The design modifications of navigation training structures include stepped-up dikes, notched rock dikes, revetments, off bank line revetments, chevron dikes, notched closure, hard points in side channels, and bend way weirs as shown in Figure D-117.

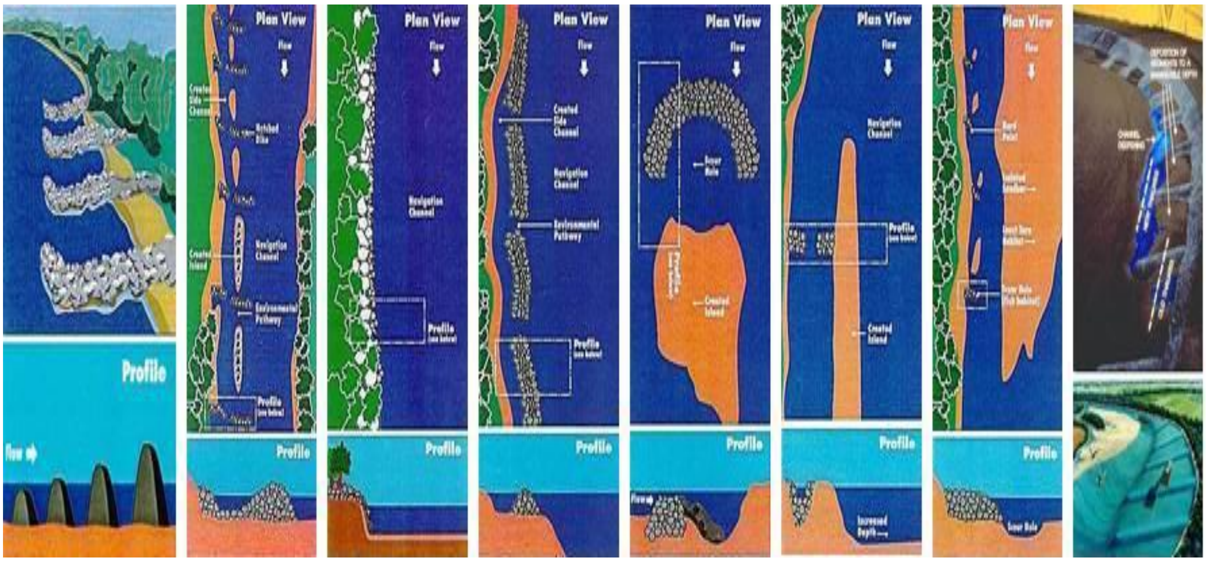


Figure D- 117 River navigation training structures.

a) *Stepped-up dikes:* Stepped-up dike fields of various elevations provide an additional element of diversity. They counteract sediment deposition, thereby preventing the conversion of aquatic environment into terrestrial. This design utilizes the river‘s energy to change the sediment deposits as the water level rises and falls. When the river‘s current hits the first dike it is propelled toward the main channel. As the river level rises, it moves over the first dike and hits the second dike, once again moving back into the main channel. This process repeats itself as the river rises and falls. The river‘s current, moving over each submerged dike, allows the sediment buildup to be redistributed back into the main channel and carried downstream.

b) *Notched rock dikes*: Notched dikes, running perpendicular to the shore, are used to guide the river and maintain the navigation channel. The river is allowed to move in and out between the notches creating all four of the primary river habitats (fast water, slow water, quiet water and wetted-edge). Sediment buildup forms small sandbars between each of the dikes. A variety of notch locations, sizes and widths can be considered to create the optimum design and to create diverse habitat environments.

c) *Revetments:* Revetments are used to stabilize eroding riverbanks and involve the removal of existing vegetation followed by grading the bank to form a stable slope on which to lay rocks. The new design approach uses a different gradation of rock with a maximum size of 5,000 pounds to provide greater bank stability and to allow coexistence of trees and rock revetment and to provide greater habitat diversity.

d) *Off bank line revetments:* In areas where the caving river bank is on the shallow side of the river, by placing a parallel structure of stone off the bank line, erosion is reduced and diverse habitats are maintained. In some areas, the revetment is notched allowing fish to move between the fast water and the slow water easily. The areas between the revetments and the bank line are considered to be prime fishing locations

e) *Chevron dikes*: A navigation structure called chevron dike was developed to improve river habitat and to create beneficial uses of dredge material. These structures are placed in the shallow side of the river channel pointing upstream. When dredging is needed to improve the main navigation channel, dredge sediment is deposited behind the chevron dike. These small islands encourage the development of primary river ecosystem habitats. In addition, various microorganisms cling to the underwater rock structures, providing a food source for fish.

f) *Notched closure structures:* Side channels are not used for navigation, but are valuable environmental areas. Notching a closure structure tends to keep the side channels from being filled with sedimentation. These structures form areas of deep water and shallow water creating a diversity of habitat, attracting different species of fish.

g) *Hard points in side channels:* Hard points are very short rock dikes that are used to stabilize side channel river banks. These navigation structures extend from the riverbank into the river and do not cause a significant buildup of sediment. Their contribution to habitat improvement is the creation of scour holes under the hard-points. These deep plunge holes attract catfish that flourish in this environment.

h) *Bend way weirs*: The Bend way weir is a low level, totally submerged rock structure that is positioned from the outside bank line of the river bend and angled upstream toward the flow. These underwater structures extend directly into the navigation channel underneath passing tows. Their unique position and alignment alter the river‘s spiraling, secondary currents in a manner which shifts the currents away from the outside bank line. The weirs control excessive channel deepening and reduce adjacent riverbank erosion on the outside bend way. This results in a wider and safer navigation channel through the bend without the need for periodic maintenance dredging. The bend way weir also eliminates the need for dikes to be constructed on the inside of the bend way therefore protecting the natural beauty and habitat of this sensitive environment.

The HSR Micro Model Theory

The principle behind the use of a hydraulic sediment response model (Figure D-118) is similitude, the linking of parameters between a model and prototype so that behavior in one (model) can predict behavior in the other (prototype). There are two different types of similitude; mathematical similitude and empirical similitude. Mathematical similitude is founded on the scale relationship between all linear dimensions (geometric similarity), a scale relationship between all components of velocity (kinematic), or both geometric and kinematic similarity with the ratio of all common point forces equal (dynamic similarity). In contrast to mathematical similitude, empirical similitude is based on the belief that the laws of mathematical similitude can be relaxed as long as other more fundamental relationships are preserved between the model and the prototype [160]. The HSR (Hydraulic Sediment Response) micro modeling does not follow a stringent set of similitude law rather it focuses on the similarity between the response of the model bed and the bed of stream under study. The micro models are developed to behave like the actual river or stream through an empirical calibration or replication process. Water flow and sediment loads are first simulated on the model through a computer-controlled operation system, with bed forms such as point bars, scour holes, and crossings left to develop naturally within physical channel boundaries of the model inserts.

Further adjustments to the micro-model slope, sediment load, discharge hydrograph, and entrance and exit boundary conditions will eventually cause the micro model to reproduce a bed response similar to that of the actual river under study [154]. The HSR micro models depend on similitude in the morphologic response, i.e. the ability of the model to replicate known prototype parameters associated with the bed response in the river under study.

Bed response includes thawed (or line of maximum depth) location, scour and deposition within the channel and at various river structures, and the overall resultant bed configuration. These parameters are directly compared to what is observed from prototype surveys. Detailed cross-sectional analysis of prototype and model surveys defining bed response and bed configuration have shown that HSR micro model variation from the prototype closely approximates that of the natural variation observed in the prototype. This correspondence allows hydraulic engineers to use the HSR micro model with confidence and introduce alternatives in the model to approximate the bed response that can be expected to occur in the prototype [158,160].

Figure D- 118 A HSR model.

HSR Micro Model Setup

The HSR micro model consists of a plan form insert constructed from polyurethane foam fabricated to geo-referenced aerial photography. The insert is placed within a hydraulic flume that contains a reservoir, electronic control valves, pumps, a constant head pipe network, and flow meters, all interfaced with a computerized control system as shown in Figure D-119 [158]. Apparatus for modeling the sediment transport characteristics of a selected river reach includes an elevated inclined platform adapted to receive an insert representing a scaled model of the section of river to be studied and a water source for delivering water containing simulated sediments to the model. As the water flows over the model, the sediment is transported so as to simulate the sedimentation characteristics of the modeled portion of the river. The apparatus is provided with a function generator which allows the water to be delivered to the model in accordance with a specified hydrograph and is also provided with a sliding digital micrometer survey system which allows accurate surveys to be taken at selected increments along the model [156]. Discharge and sediment (granular plastic, urea, specific gravity 1.4) are simulated through the insert channel. A model coordinate system is established to collect all data. Lasers are used to collect detailed bathymetry and normalized velocity distribution from the model for comparison to hydrographic surveys and ADCP (Acoustic Doppler Current Profiler) profile data from the river (Figure D-120). High definition cameras are also used for flow visualization and general model observation recording [158].

Computer controlled automation combined with highly accurate measurement devices are the keys to this micro model technology. The hydraulic processes of a river or stream under study are replicated by employing a series of integrated process control valves, centrifugal pumps, micro level measurement gauges, and customized computer hardware and software. These devices allow the engineer to automatically control the flow of water and sediment through the model. The engineer is then able to allow the natural, complex hydraulic principals of moving water and sediment develop a duplicate bed form of the actual river in the HSR micro model. A high resolution three-dimensional laser scanner is then employed to collect bed topography data on the model [154].

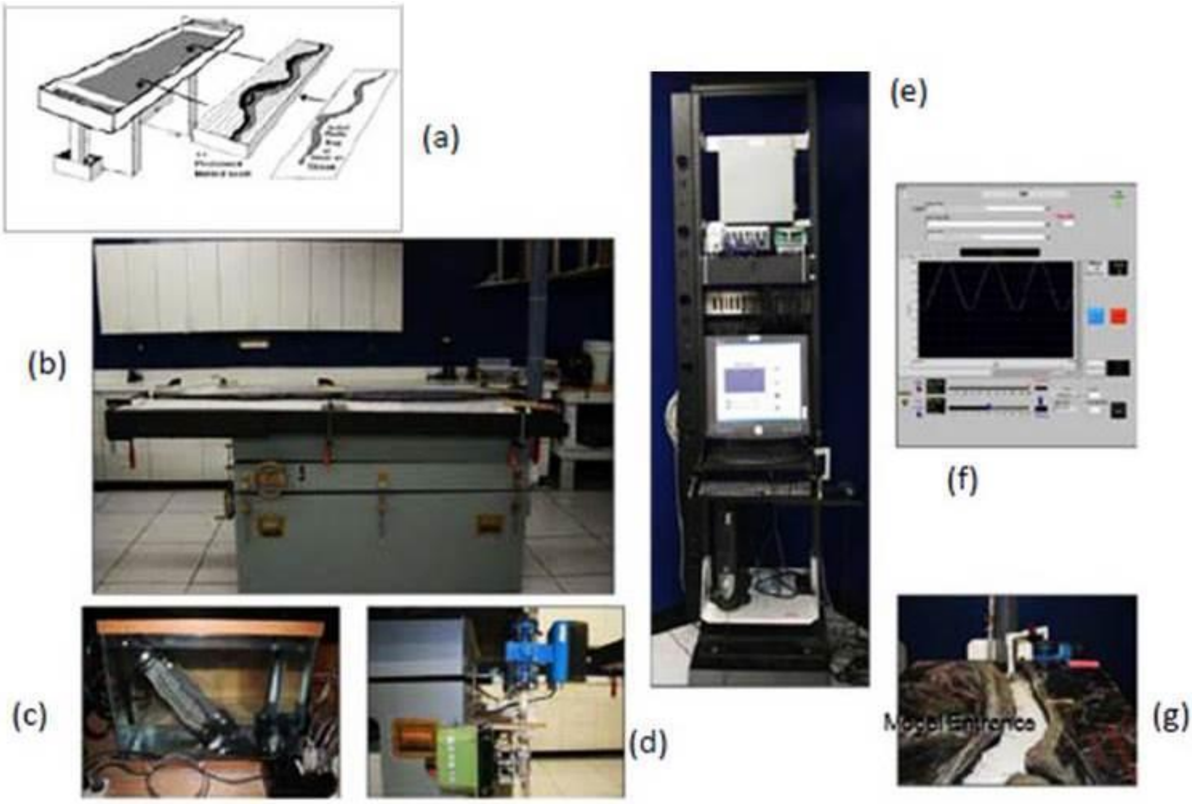


Figure D- 119 HSR model operational schematic: (a) Model insert; (b) Model flume; (c) Reservoir and pump; (d) Control valve and flow meter; (e) Computer rack; (f) User interface; and (g) Model insert with sediment.

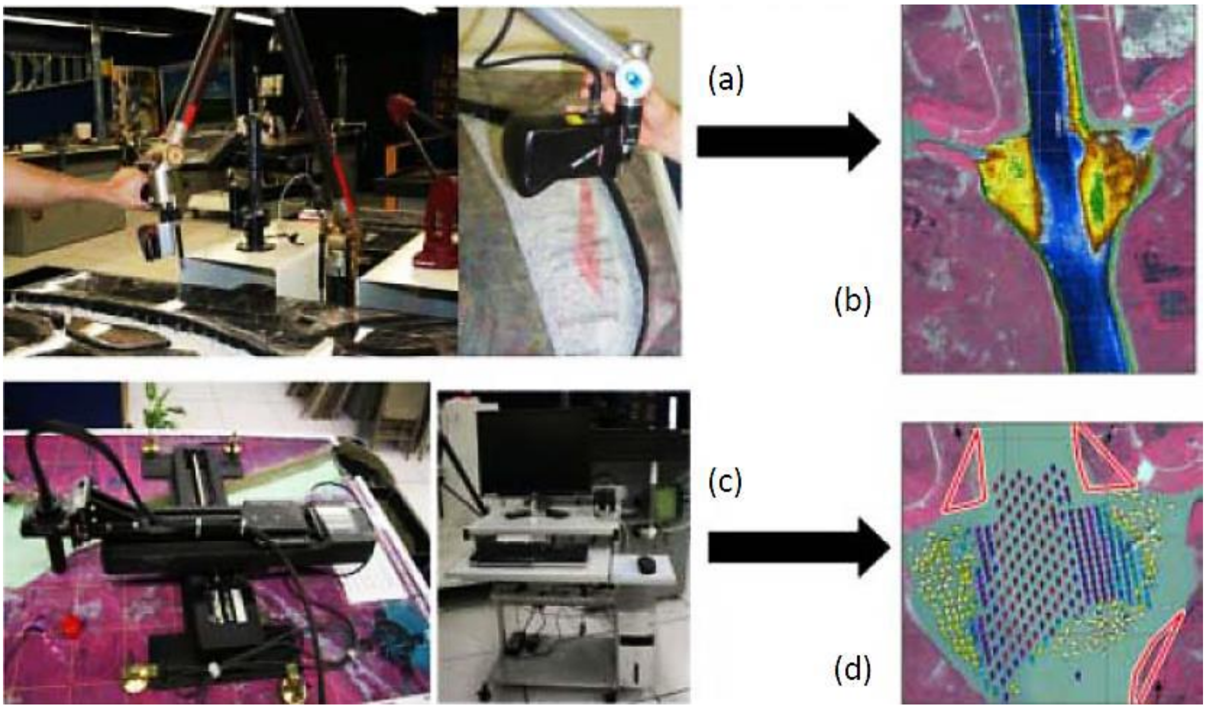


Figure D- 120 HSR model basics: (a) Laser scanner; (b) Laser bathymetry; (c) Laser Doppler Velocimeter (LDV); and (d) LDV normalized velocity vector output.

The Modeling Procedure

The HSR modeling methodology employs a calibration process designed to replicate the river‘s loose boundary condition at the time of the model study (Figure D-121). Replication is defined as the ability of the model to reproduce the mobile bed response of the river. It is achieved during model calibration and involves a three-step process. First, fixed planform boundary conditions of the study reach, i.e. bank lines, islands, side channels, tributaries, and other features are established according to the most recent available high-resolution aerial photographs and topography. Various other fixed boundaries that exist in the river are also defined including river training structures, submerged rock, consolidated clay, and other non-mobile boundaries. Second, loose boundary conditions of the model are developed. Synthetic channel bed material is introduced throughout the model in an arbitrary amount to approximate level plane. Third, steady state discharge simulation tests are run through the model. Discharge, sediment volume, model slope, fixed boundaries, and entrance conditions are refined during testing as part of model calibration. The model bathymetry is developed from a static, flat bed into a fully-formed, dynamic, three-dimensional bed response [154].

These models are distorted linearly (horizontal to vertical scale) to generate sufficient forces necessary for bed movement. Distortion has varied from as small as 6 to as large as 22. The HSR model relies on hydrodynamics and sediment transport to develop its own equilibrium bed response and resultant three-dimensional bed configuration within the channel. The calibration process of the HSR micro model involves adjustments in the flow, model slope, model entrance conditions, model vertical scale, sediment volume, and fixed boundary conditions.

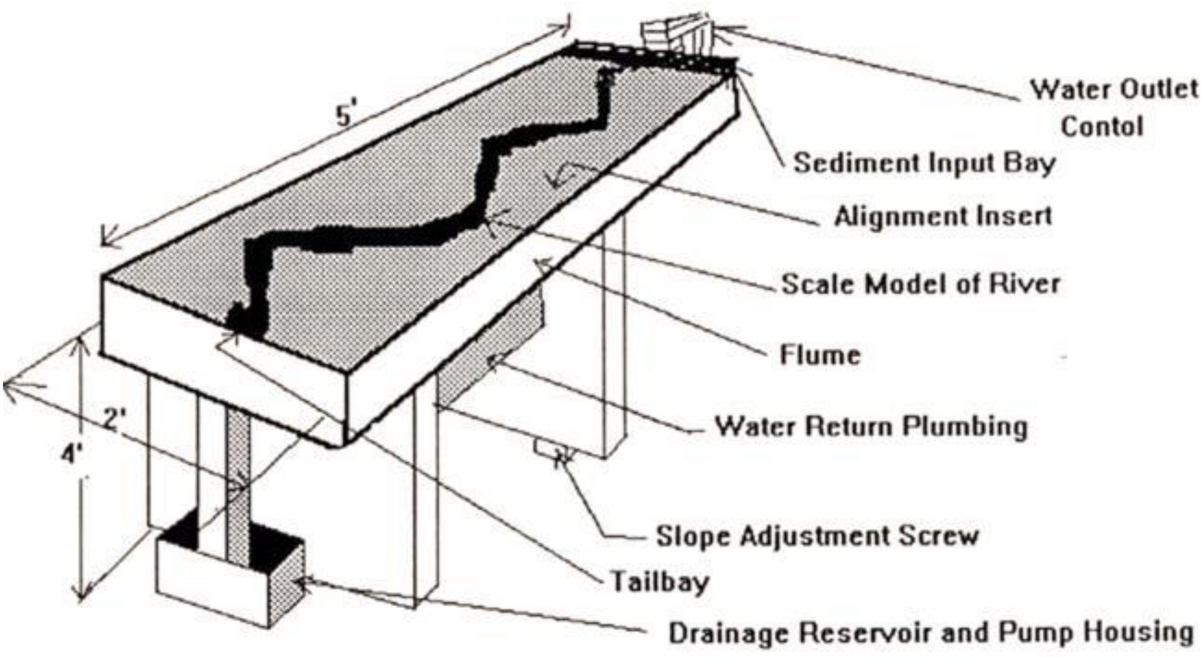


Figure D- 121 A typical micro river model.

1. Model Calibration and Replication

The HSR modeling methodology employs a calibration process designed to replicate the general conditions in the river at the time of the model study. Replication of the model is achieved during calibration and involved following three step process [160].

a. Planform "fixed" boundary conditions of the study reach, i.e. bank lines, islands, side channels, tributaries and other features are established according to the most recent available high resolution aerial photographs. Various other fixed boundaries are also introduced into the model including any channel improvement structures, underwater rock, clay and other non-mobile boundaries. These boundaries are based off of documentation (such as plans and specifications).

b. "Loose" boundary conditions of the model are replicated. Bed material is introduced into the channel throughout the model to an approximate level plane. The combination of the fixed and loose boundaries serves as the starting condition of the model.

c. Model tests are then run using steady state discharge. Adjustment of the discharge, sediment volume, model slope, fixed boundaries, and entrance conditions are refined during these tests as part of calibration. The bed progresses from a static, flat, arbitrary bed into a fully-formed, dynamic, three-dimensional mobile bed response. Repeated tests are performed for the assurance of model stability and repeatability. When the general trends of the model bathymetry are similar to observed recent river bathymetry, and the tests are repeatable, the micro model is considered calibrated and alternative testing can begin.

d. One important parameter to note in some cases is that in calibration, non-erodible bed material (clay) is used in a localized area on the model riverbed to represent the bluffs and rock outcroppings of the bank line. Because the non-erodible material is required for calibration, the non-erodible remained in the model throughout the rest of the study (i.e. during alternative testing).

2. Scales and Bed Materials

Typical scale for micro-models have ranged between 1:15,000 and 1:600 horizontal and 1:1,200 and 1:100 vertical, with distortion ranging between 5 and 13, Distortion is necessary in most physical models to sufficiently move the sediment [157]. This distortion supplied the necessary forces required for the simulation of sediment transport conditions similar to those observed in the prototype. The bed material such as granular plastic urea (Type II) with a specific gravity of 1.40 may be used.

3. Appurtenances

The HSR model planform insert is constructed according to the high-resolution aerial photography of the study reach. The insert is then mounted in a standard HSR model flume. The riverbanks of the model are routed into dense polystyrene foam and modified during calibration with clay and polymers. Rotational jacks located within the hydraulic flume controlled the slope of the model. River training structures in the model are made of galvanized steel mesh to generate appropriate scaled roughness.

4. Flow Control

Flow into the model is regulated by customized computer hardware and software interfaced with an electronic control valve and submersible pump. This interface is used to control the flow of water and sediment into the model. For all model tests, flow entering the model is held steady to serve as the average expected energy response of the river. Because of the constant variation in the river, this steady state flow is used to replicate existing general conditions and empirically analyze the ultimate expected sediment response that could occur from future alternative actions.

5. Data Collection

Data from the HSR model is collected with a three-dimensional laser scanner. The river bed in the model is surveyed with a high definition 3-D laser scanner that collects a dense cloud of xyz data points. These xyz data points are then geo-referenced to real world coordinates and triangulated to create a 3-D surface. The surface is then color coded by elevation using standard color tables that are also used in color coding prototype surveys. This process allows a direct comparison between HSR model bathymetry surveys and prototype bathymetry surveys.

6. Replication Test

Once the model adequately replicated general prototype trends, the resultant bathymetry serves as a benchmark for the comparison of all future model alternative tests. In this manner, the actions of any alternative, such as new channel improvement structures or realignments can be compared directly to the replicated condition. General trends are evaluated for any major differences positive or negative between the alternative test and the replication test by comparing the surveys of the two and also carefully observing the model while the actual testing is taking place. Bathymetric trends are recorded from the model using the 3-D Laser scanner. Calibration is achieved after numerous favorable bathymetric comparisons of the prototype surveys are made to several surveys of the model.

7. Design Alternative Tests

The testing process consists of modeling alternative measures in the HSR model followed by analyses of the bathymetry and velocity results. The goal is to identify the most effective and economical plan to reduce the dredging. Evaluation of each alternative is accomplished through a qualitative comparison to the model replication test bathymetry (deposition) [160].

Once the engineers achieve the replication, they can use the micro-model to make qualitative assessments about the effect of prospective design alternatives. They can realign channels or install such additional structures as dikes and underwater weirs within the micro-model. Through computer software, users of the micro model can adjust the discharge and corresponding sediment load at any time during the simulation by a simple click of the mouse. Controlled automation and highly accurate measurement devices are the keys to this technology. The micro models use integrated process control valves and durable centrifugal pumps to emulate a repeatable discharge/sediment response through the model channels. They are engineered to automatically run hydrographic simulations over any given time sequence. An important feature of micro modeling is its dynamic sediment and flow equilibrium, which is obtained by a simple but well-designed reservoir and sediment filter chamber system contained under the tabletop model [154].

An Example of River Sedimentation Study using Micro Models

Solving a chronic dredging problem on the Atchafalaya River

This study encompassed the Berwick Bay reach, which is located on the Atchafalaya River (the largest of all distributaries of the Mississippi River) between Morgan City and Berwick, in southern Louisiana. Changing flow patterns throughout the years have steadily increased the depositional rate. This particular stretch of river has been one of the most troublesome reaches on the Atchafalaya River in terms of dredging cost, frequency, and volume. Repetitive maintenance dredging are required at this location approximately twice per year. Severe deposition had been experienced at the harbor facilities adjacent to Morgan City. In 1999, the USACE New Orleans District initiated a study to examine a possible structural solution to the dredging problem. The District enlisted the help of the Applied River Engineering Center of the St. Louis District to model this reach using micro modeling technology. The study was carried out to evaluate the current sediment and flow response trends through this problem area. The model (known as Berwick Bay micro model) was then used to determine the design and placement of several underwater weirs that would lessen the impact of the sediment and flow problems experienced at this location [161].

Berwick Bay micro model: The model (Figure D-122a) used for this study was constructed according to the high-resolution aerial photograph of the study reach shown in Figure D-122b. The scales of the model were 1 inch = 600 feet, or 1:7200 horizontal, and 1 inch = 100 feet, or 1:1200 vertical, for a 6 to 1 distortion ratio. This distortion supplied the necessary forces required for the simulation of sediment transport conditions similar to those of the prototype. The bed material used was granular plastic urea (Type II) with a specific gravity of 1.4 [161].

Figure D- 122 a) Berwick Bay micro hydraulic model; and (b) Aerial photograph of the Atchafalaya River at Morgan City and Berwick, Louisiana.

In all model tests, an effective discharge or hydrograph was simulated in the Atchafalaya River channel. This hydrograph served as the average design energy response of the river. Because of the constant variation experienced in the prototype, this hydrograph was used to theoretically analyze the ultimate expected sediment response. Each hydrograph simulated a discharge range between extreme low-flow to high "within-channel" flow. Flow rates in the model ranged between 0.85 to 1.35 gallons per minute. The most important factors during the micro modeling process were an equilibrium condition of sediment transport and the simulation of high and low energy conditions. High flow in the model simulated a peak energy condition representative of the river's bed forming flow and sediment transport potential at bank full stage. The time increment or duration of each hydrograph cycle (peak to peak) was two minutes.

Calibration and verification of the micro model: The calibration and verification of the model involved the adjustment of water discharge, sediment volume, hydrograph time scale, model slope, and entrance conditions of the model. These parameters were refined until the measured bed response of the model was similar to that of the prototype. Data available from the prototype used for the calibration process included several hydrographic surveys, ADCP velocity data, aerial photographs, and on-site field reconnaissance. Model calibration was achieved once a favorable comparison of the prototype surveys was made to several surveys of the model. The base test was developed from the simulation of successive repeatable design hydrographs until bed stability was reached and a similar bed response was achieved as compared with prototype surveys. This survey then served as the comparison bathymetry for all design alternative tests (see Figure D-123). The resultant bathymetry of this bed response served as the base test of the micro model. Figure D124 shows the flow visualization photo of the base test, which served as the comparison flow patterns for all design alternative tests. The trends of the model were very similar to the prototype velocity vectors established from the ADCP data. The model demonstrated that most of the flow was concentrated along the right descending bank and to the right of the navigation spans which are located in the center of the channel.

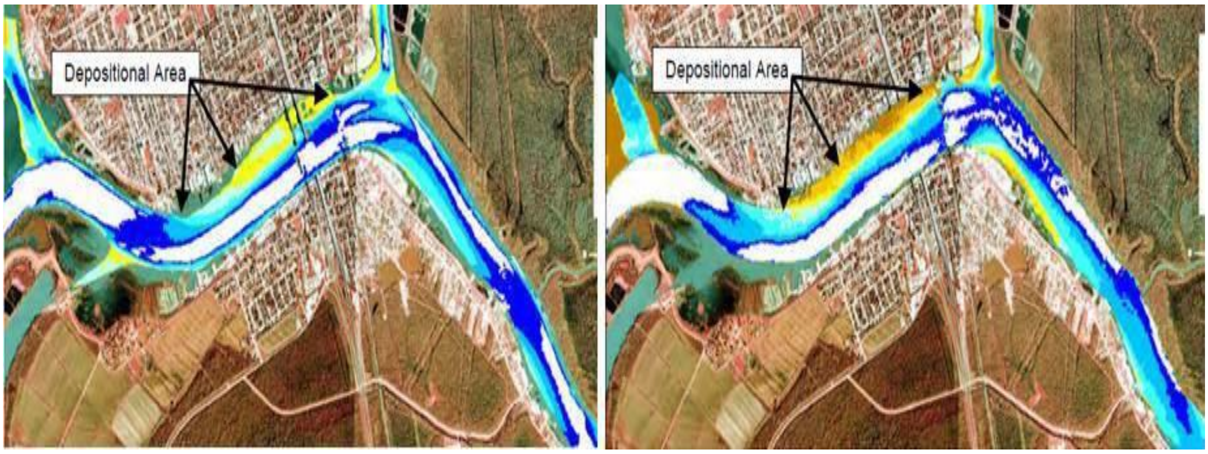


Figure D- 123(a) Prototype survey used for model calibration; (b) the resultant bed configuration of the micro model base test.

The only river training structures model tested to solve the problems in the Morgan City/Berwick Bay reach was of the underwater variety. It was required by the study that any design solution could not restrict vessel movement between both bank lines of the river. Therefore, traditional dike structures were not considered feasible.

All the weir designs studied in the micro model were tested at depths suitable for the passage of barge traffic at all river stages. The micro model indicated that the most effective design to solve the problems consisted of 10 bend way weirs located within a one-mile reach of river and at a depth of 20 feet below the low water stage. The resultant flow patterns and bathymetry developed by this design in the model are shown in Figures D-124 (b) and D-125.

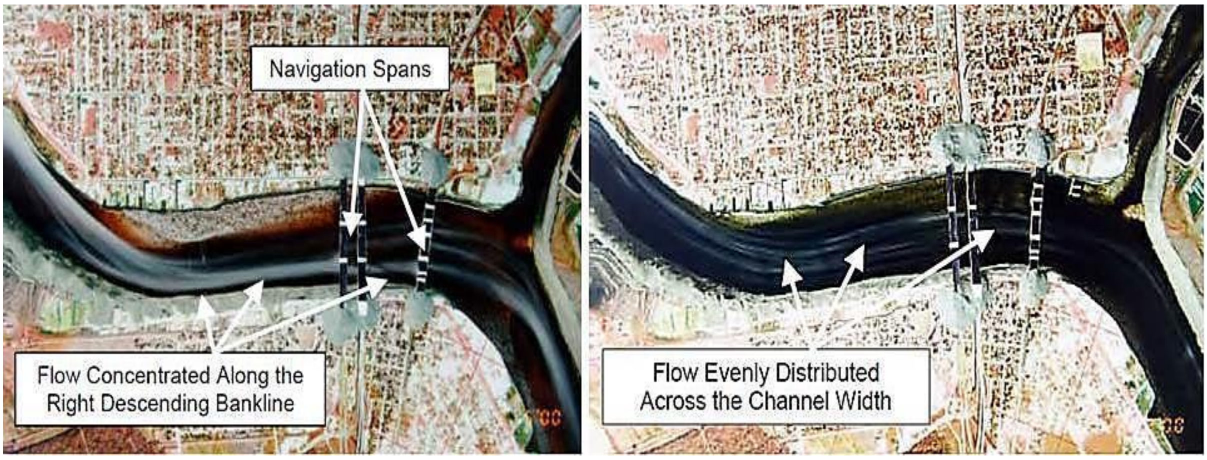


Figure D- 124(a) Micro model base test flow visualization; (b) Flow patterns developed by bend way weirs in the alternative design.

The results demonstrated that the design proved very effective at removing a substantial portion of the depositional area along the left descending bank line. The design also completely shifted the thawed towards the center of the channel at the upstream portion of the reach. The weir field effectively created a smooth transition of the thawed from the bend towards the middle of the channel and into the straight reach upstream of the bridges. The flow visualization photos demonstrated a significant redistribution of flow across the channel width. The design indicated that the flow patterns were more evenly distributed across the entire channel width and were no longer concentrated along the right descending bank line. This change in flow patterns decreases the dangerous currents that effect down-bound tows navigating through the bridge openings [161].

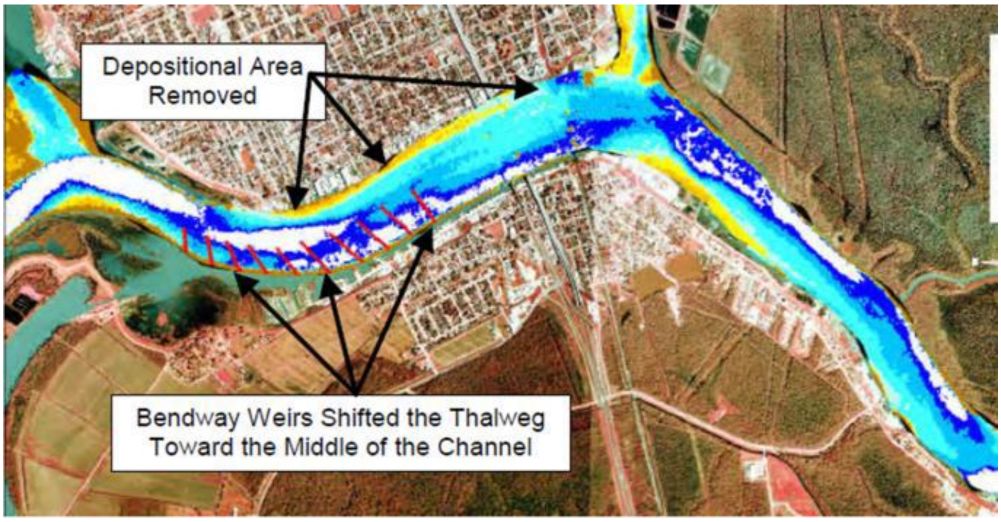


Figure D- 125 Bathymetry developed by bend way weirs in the alternative design.

Demonstration Micro Hydraulic Models

Micro models are effective demonstration tools for engineers and managers to communicate with the public concerning river projects. They have also become useful tools for science, education and conservation. A pioneering use of portable demonstration hydraulic models can be found in a 1960 publication by the USDA Agricultural Research Service entitled "Let's Demonstrate Hydraulic Phenomena: A Guide for the Operation and Use of the Portable Demonstration Channel and Models of Hydraulic Structures". Such portable models were used to demonstrate performance of hydraulic structures such as pipe entrances and outlets and open channel hydraulics including hydraulic jumps and flow on an adverse slope. As shown in Figure D-126 the portable model was a stand-alone unit equipped with control panel, pump and reservoir [165].

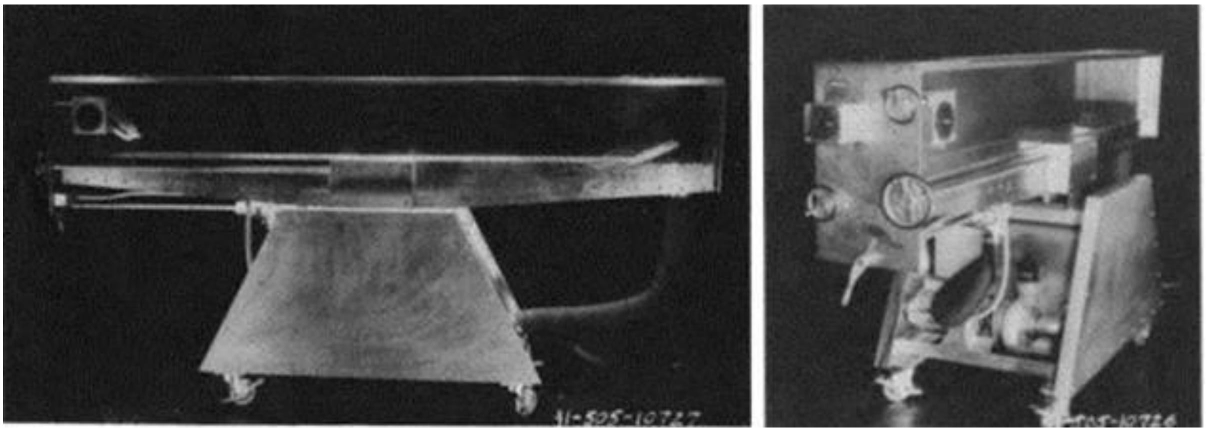


Figure D- 126 USDA hydraulic demonstration channel and the control panel, pump and reservoir.

The U.S. Federal Highway Administration (FHWA), when conducting training courses on highway hydraulics, uses a portable hydraulic flume in the classroom for the participants to observe numerous hydraulic principals related to highway drainage design. The participants take velocity and discharge measurements from the flume while in various setups and use the information to make design calculations [166]. Mini demonstration hydraulic flumes (Figure D-127) have also been used by a charity organization, JBA Trust, to promote knowledge and skills in managing floods. The mini hydraulic flume is a simple channel, driven by a system of recirculating pumps. It includes scale models of hydraulic structures such as weirs, bridges, culverts and debris screens. It has been used to promote understanding of some of the causes of flooding and how good design and maintenance of rivers and drainage channels can help to manage flood risk [167].

Figure D- 127 Performing field demonstrations using mini hydraulic flumes.

Today's commercially available micro demonstration models are more sophisticated because they are capable of simulating fluvial processes and are available to scientific and educational institutions and public agency. A company, Little River Research & Design, has developed several types of micro river model for applications in the fields of science, education and conservation (Figure D-128). Their Emriver models [163] are capable of simulating complex fluvial, hydrologic, and geomorphic processes using robust and highly engineered components and their unique plastic modeling media. The Emriver models are designed and instrumented as effective, cross-disciplinary research and teaching tools that:

- Foster STEM (Science, Technology, Engineering, and Mathematics) education through innovative hands-on learning.

- Immerse students in scientific methodology and observation.
- Engage the public to broaden conservation outreach.
- Enhance research in geomorphology - including sedimentology and coastal processes.
- Demonstrate fluid mechanics and river habitat hydraulics.
- Facilitate best practices in civil engineering and landscape architecture.

Figure D- 128 An Emriver model and a demonstration session.

Emriver river models: Currently four models are available from Little River Research & Design:

1. Emriver Em4: This is a robust, modular research system capable of a wide array of simulations such as floodplains, deltas, groundwater processes, and sediment-transport.
2. Emriver Em3: A modular and portable system designed for research and teaching of river dynamics.
3. Emriver Em2: This model is optimized for ease of setup, use, and portability; perfect for demonstrating river science and conservation principles to audiences of all ages and educational backgrounds.
4. Emriver Emflume1: A turnkey, portable, desktop flume for studying fluid mechanics, river habitat hydraulics, and sediment transport.

Instruments and Accessories of Emriver models: Accessories, instruments, and media are designed to help achieve educational, outreach, and research goals.

- Modeling media: At 60% the density of quartz sand, company‘s unique, recycled thermoset plastic media allows for accurate scaling of river behavior, channel morphology, and sediment transport. Color-coding (Figure D-129) allows for visualization of sediment transport and deposition according to grain size. The media will not damage pumps or components.

Figure D- 129 Modeling media for Emriver models.

- K500 digital flow controller: The K500 (Figure D-130 a) provides precise control of flow from 25 to 210 ml/sec, meters, and displays current flow rate in ml/sec, shows accumulated run time and flow in liters, and runs pre-programmed hydrographs.

Figure D- 130(a) K500 digital flow controller; (b) Adjustable tilting base; and (c) Wave maker.

- Tilting base: A tilting base (Figure D130 b) can be added to adjust one or both axes of the table to change river valley slope, which expands the capabilities of the models, including the study of sedimentology, delta geomorphology, and tectonic influences on river form.
- Wave maker: Computer control of a paddle (Figure D-130 c) allows precise adjustment of wave frequency and strength to study coastal geomorphology including longshore drift, sediment delivery from river mouths, and, with the color-coded media, particle sorting by these processes.

Figure D- 131(a) Groundwater control system; (b) Media feeder; and (c) Dye injector.

- Groundwater system: An upstream spray bar and downstream extraction valves (Figure D-131a) produce parallel and uniform groundwater flow lines in the model and allow groundwater and channel flow to be known and partitioned, facilitating research on groundwater/surface water interactions.
- Media feeder system: Up to four hoppers (Figure D-131b) allow precise input of various grain sizes into the flow for studying sediment disturbances and equilibrium in fluvial systems.
- Dye injector: Programmable dye pulses (Figure D-131c) allow for better visualization of water movement and depth and are especially useful in time-lapse recording of experiments.

Figure D- 132 Vermont Department of Environmental Conservation using an Emriver micro model.

The Vermont Department of Environmental Conservation has had the Emriver stream models for more than a decade, and it remains one of their top teaching tools for both professional and lay-person trainings (Figure D-132). Their Rivers Program supports 5 stream tables used around the state as part of department‘s education and outreach work. The Emriver stream table provides a quick and easy way to share real life examples and demonstrations of river dynamics and the interactions between the rivers and what is put in and along them. This has allowed for more awareness and in-depth discussion in the community planning efforts around such subjects as: floodplain protection versus development, habitat resources, water quality concerns, and long-term goals for maintaining a healthy river system [163].

APPENDIX E- OPERATIONAL SAFETY OF HYDROMECHANICAL EQUIPMENT

Abstract

The failure of a gate at Folsom Dam occurred in California (USBR, 1995) due to corrosion of the steel trunnion pins, caused worldwide concern as to evaluate some aspects of gates seldom evaluated before. Some other important failures of gates have been caused by different mechanisms, the diversion tunnel of Tarbela Dam in Pakistan, collapsed due to cavitation when the control gate became stuck during the construction phase in 1974 (Kenn and Garrod, 1981) .Gate reliability shows a more important trend in potential failure modes as age of gates and lack of maintenance in the installations is increasing. Spillways gates provide with a vital function of flow controlling releasing from large hydraulic structures. Flow should be properly controlled as gates are operated to guarantee the safety of the spillway and of the dam itself. In the Appendix E aspects of the Operational Safety of the Hydromechanical equipment are outlined and presented to provide the readers, engineers of Dam safety, the different aspects considered to minimize such events to occur. Drip guidelines also provide with sound and useful information for operators and gate maintenance in the Manual for Structural Assessment.

.

Overview

Failure of a spillway has potentially serious consequences, which can include loss of reservoir storage, loss of life, and downstream damage. Potential failure modes for spillways include: flows exceeding spillway discharge capacity, cavitation damage, hydraulic jacking, foundation erosion, loss of the reservoir / head cutting due to failure of spillway chute or stilling basin etc. In addition to this, improper gate operation and failure of gates could also result in a spillway failure. Although the safety of a dam is evaluated thoroughly, spillway gates and associated equipment often receive less attention than other features. In many cases failure of individual gates has not resulted in serious consequences but the possibility exists if breakdown occurs at a critical time of flood release as failure of a single gate can be followed by failures of other gates.

As per ICOLD data 40 per cent of failures are due to overtopping of Dam / spillway gates and none / late operation of spillway gate which is significant cause of overtopping of dam other than inadequate capacity of its spillways & spillway blockage with debris causing the dam to be overtopped. Further failure of gate and structures in its vicinity can occur due to overstressing of gate and hoist parts which in turn can arise from overtopping or due to other reasons like seismic load, excessive increase in reservoir level, and vibration during overtopping, etc. Therefore correct design, proper construction and thereafter regular maintenance with reliable operation of the spillway gates are equally critical to assure safety of the dam and downstream residents.

Appendix E will illustrate the potential causes of failure of spillway gates and measures to improve the reliability of gate operation.

Reasons of failure of Gates& allied structures

Potential reasons of failure of gates are:

a) Structural /Mechanical / Electrical failure of Gate & allied structures or their components:-

This can be due to:

- Design Inadequacies for expected design loading and under severe condition including seismic, vibration, ice formation, silt accumulation, increase in reservoir level.
- Failure of parts/equipment due to aging, corrosion, erosion, lack of maintenance / lubrication of parts, quality measures /control in material and failure of power supply.
- Insufficient power or hoisting against silting and jamming of gates.

b) Operational Problems

These can be due to:

- Absence of operator.
- Wrong judgment of inflow ,incorrect action under stress.
- Absence / damage of proper access to control room or site.
- Lack of advance planning to access site.
- Failure of communication and instruments for transmitting data.
- Absence of instructions from higher authority.

- Insufficient training to operator under extreme flood condition.

Failure of Gates & allied structures can be broadly classified as:

1. Structural Failure.
2. Mechanical Failure.
3. Electrical Failure.

1. Structural Failure

Structural Failure is mainly due to following reasons:

- Inadequate Design.
- Excessive Friction.
- Failure of gate leaf and other parts due to erosion / corrosion.
- Failure of arms & trunnion assembly (in radial gate), bearing, Girders and stiffeners.
- Failure of Gate Anchorages.
- Failure of Hoist / hoist support structure.
- Failure of Lifting Bracket at gates.
- Lack of access, icing (freezing of gate and gate parts), weathering/any other severe condition.
- Vibration of gates / gate parts.
- Accumulation of Debris in waterway.

2. Mechanical Failure

Mechanical Failure of gate is mainly due to the following reasons:

- Inadequate Design.
- Failure of Driving Mechanism (Motor, Gears, coupling, bearings, shaft, Brakes).
- Failure of Hydraulic cylinder parts and power unit component (for Hydraulic Hoist).
- Failure/seizure of Lifting component i.e. chain, wire rope, socket and sheaves.
- Failure of parts due to lack of lubrication and Painting.
- Failure of Hoist due to Jamming of gates.
- Seal leakage and gate operation problem due to ice formation / silt accumulation.

3. Electrical Failure

Electrical Failure of gate is mainly due to following reasons:

- Inadequate Design and rating of equipment/parts
- Failure of Motor / pump
- Failure of Control circuit component
- Failure due to limit switch
- Failure of Power supply

- Condensation of Electrical components / contacts.

1. Structural Failure

a. Design

An Example of Structural Failure (Figure E-1):

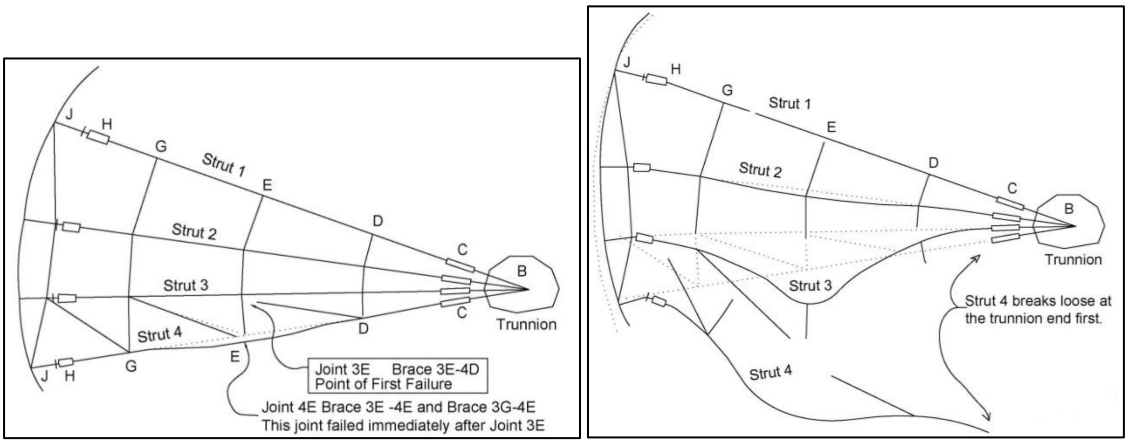
Structural failures in gates are relatively rare but the spectacular failure in July 1995 of Spillway Gate No. 3 of the Folsom Dam in California due to excessive pin friction caused by corrosion of the steel trunnion pins and insufficient stiffness / strength in critical structural gate arm members has led to worldwide examination of similar undetected problems.

The radial gates were not designed for any trunnion friction load and presence of high friction at trunnion due to corrosion of trunnion pins made up of carbon steel (prone to early corrosion) resulted from of reduced frequency of lubrication and lack of weather protection (allowing ingress of water /water vapor inside hub and at moving parts) contributed to increase of tension in the brace member in the right arm frame lead to failure of bolted connection at upper end of brace. Once the initial diagonal brace failed, load was transferred to the adjacent deg) results high tension forces & bent type arm at trunnion end led to second order bending & torsion in arm struts.

In addition to hydrostatic load with other load like self-weight, ice / silt load, wave affect etc., gate geometry and imperfections of gate arms may introduce second order forces lead to deformation of gate girders which may bend the arm struts in the out-of-arm frame plane.

Imperfections in the assembly of the gate structure together with deflection of the arm struts caused by the self-weight may result in eccentricities of the arm struts. The eccentricity will be magnified by the compression forces in the arm struts increasing the bending moment in the arm in the plane of arm frame. These eccentricities will lead to increased second order bending moment in the struts due to the axial compressive load from the reservoir.

Spillway radial gates transfer the reservoir load to the trunnion pin through compression of the relatively slender gate arms. Spillway Crest Gates are normally operated with the reservoir water surface below the top of the gates to pass normal flows during maintenance and routine exercising of the gate and does not specifically recommend operation with water surface above the top of the gates. Though overtopping over the gate is not allowed as per standard but Spillway gate overtopping conditions should be considered if it has a reasonable chance of occurrence. Top sealing radial gates are submerged and can be designed for expected water head.



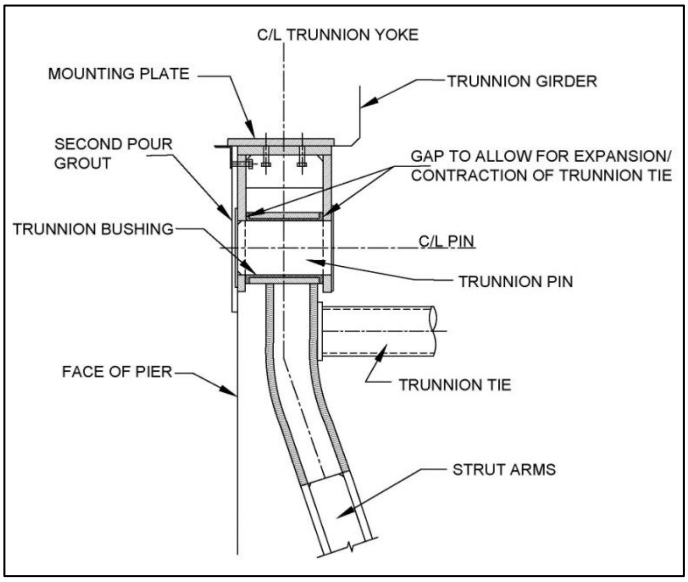


Figure E- 1 Failure of Gate of Folsom Dam, California, USA

Brace connections, which failed in turn. Immediately following the brace connection failures, the lower most struts (strut No. 4) buckled downward leading to buckling of the remaining arm struts. Other factors contributed to this failure are large diameter of pin (32 inch hollow pin) causing large trunnion friction movement at trunnion and higher bending moment at gate arm , small angle between brace member and arms (14 deg) results high tension forces & bent type arm at trunnion end led to second order bending & torsion in arm struts.

Spillway radial gates transfer the reservoir load to the trunnion pin through compression of the relatively slender gate arms. Spillway Crest Gates are normally operated with the reservoir water surface below the top of the gates to pass normal flows during maintenance and routine exercising of the gate and does not specifically recommend operation with water surface above the top of the gates. Though overtopping over the gate is not allowed as per standard but Spillway gate overtopping conditions should be considered if it has a reasonable chance of occurrence.Top sealing radial gates are submerged and can be designed for expected water head.

In addition to hydrostatic load with other load like self-weight, ice / silt load, wave affect etc., gate geometry and imperfections of gate arms may introduce second order forces lead to deformation of gate girders which may bend the arm struts in the out-of-arm frame plane.

Imperfections in the assembly of the gate structure together with deflection of the arm struts caused by the self-weight may result in eccentricities of the arm struts. The eccentricity will be magnified by the compression forces in the arm struts increasing the bending moment in the arm in the plane of arm frame. These eccentricities will lead to increased second order bending moment in the struts due to the axial compressive load from the reservoir.

The maximum moment can be expected to occur when the gate is starts to open to regulate the reservoir level, the hoist loads and the trunnion pin friction loads are mobilized, magnifying the bending of the gate arms and related second-order forces / bending moment. Further frictional resistance due to lateral trunnion reaction (force parallel to the axis of gate trunnions) between arm hubs and the trunnion yoke if any, increase the bending moment of the gate arms affecting the stability of the gate due to overstressing which increases the combined stresses in gate arms.

Normally magnitude of such moment is relatively small as to affect bulking of arms but due to non-maintenance / absence of lubrication at trunnion bearing, excessive friction may lead to such failure especially for aged gates.

For radial gate arm buckling, the combined stress ratio are to be checked for the axial compressive stresses with the vertical and horizontal bending stresses which act at the extreme fibers at the same cross section of the arms.

Other factors that contribute to the potential for structural failure include corrosion of the critical gate members and their connections, overtopping the gate during flood events .Significant spillway pier deformation, improper modifications to gate structure (gate height rising, welded new components etc.) Ice forming on the gate structure, uneven lifting loads, fatigue of structural gate members, etc. Excessive vibration of the gate during operation can lead to increased cycles leading to fatigue cracking.

Structural analyses of the gate structure are performed to evaluate the stability and the stresses levels in the gate members under combined reservoir and extreme loadings (seismic). The analysis for the strength and stability should include axial, shear, and bending deformation in the structural members and their connections. In general, it is required that stability is maintained for the gate as a whole and for each component of the structure. Calculation by Finite element method may be performed in lieu or addition to manual calculation to evaluate gate performance and risk assessment for large / complex structure. For preliminary analyses, only the arm struts may be modeled. For higher level analyses, all gate members including the gate leaf, girders, and gate arms would be modeled. When analyzing the gate structure with extreme loading (flood / seismic conditions), the following should be considered.

a) Initial imperfections of the gate assembly – imperfections considered at the points of intersection of arm struts and their brace members.
b) Initial deformation of the gate arms due to the gravity load.
c) Stiffness reduction due to inelasticity.
d) Deformation of gate arms by the bending moment due to deflection of the gate girders.
e) Defects in the gate members and their connections.

b. Seismic

Lack of consideration of seismic loading may be the most subtle of these deficiencies. First consideration of seismic loadings frequently occurs when the design of gated spillways is revisited during a process of rehabilitation of the spillway. For older dams seismic loads may have not been considered in design but later observed that probability of severe earthquakes may occur and earthquake damage to gates could be significant. For examples of failure of gates due to earthquake are as under:

- In 1990 a magnitude 7.3 earthquakes in Iran caused serious damage to the radial gate & experienced Accelerations of 6.09 were significantly greater than design value. The dynamic load exerted on the radial gates caused trunnion arms to buckle even though the spillway water level in the reservoir was 6 meters below maximum normal level.
- In Radial Gates, concentrated loads transferred to the pier via the trunnion will be a function of the gate size, the reservoir level especially at the time of the earthquake / seismic loads.Even if the gates do not concentrate loads on the spillway piers, the inertial effects on the spillway piers during earthquake loads could be significant.
- In radial gates, trunnion anchorage is typically provided in spillway piers to anchor the trunnion. During earthquake loads, the anchorage may be over stressed and hence anchorage should be evaluated for seismic loads.
- Spillway gates are typically anchored within the spillway piers, which imposes additional loads into the piers during an earthquake. On earthquake, any failure of the spillway piers or excessive pier deflection can directly lead to gate failure if the gates lose their support or load the gates laterally and cause them to buckle. A single pier failure may also result in failure of the two adjacent spillway gates.
- In some cases, Deck Bridge may also support the hoisting equipment / crane used to operate the gates. Such spillway gates could be evaluated for potential impact loading from the failed bridge if so on extreme / seismic loads.

c. Vibrations

Example for Vibration Failure:

Collapse of a spillway gate at the Wachi Dam (Yano, 1968) due to vibration was considered a special case due to eccentricity of the trunnion bearings.

The second most common case of design errors is failure to consider the possibility of severe vibrations of a gate which can result in structural damage or restrict operation at certain gate openings. Vibration of a gate will occur under specific/ unsteady hydraulic conditions which create hydraulic disturbance and oscillations to structure. Gate vibration may also arise due to maintenance and servicing problems, leakage, inadequate venting,and cavitation or to degradation over time. Unsteady flow conditions can also be present due to shape of spillway, piers, gate slots and accumulation of debris near gate.

In general, vibration is a result of resonance where the frequency of a pulsating force is equal or nearly equal to the natural frequency of a flexible part of the structure. The natural frequency of a gate depends on the rigidity per unit mass. When oscillation occursthe water at the gate face moves with it (the added mass). The natural period of vibration of a gate depends on its total mass (structural mass plus added mass), it will vary with the gate opening and the upstream water level. Any force due to the excitation frequency but in opposition is a damping force. At radial gates damping is due to the friction of the side seals and is relatively low. At vertical lift gates additional forces are caused by bearing friction at the guide rollers and, to a lesser extent, roller friction. Vertical slide gates does not vibrate generally even in partial opening due to high friction forces between sliding surfaces. Gate opening and upstream water level will also affect damping but not necessarily in the same ratio as affected to natural frequency. Therefore it is difficult to predict the conditions and magnitude of gate vibration hence important that design will take care to avoid such features cause fluid excitation.

One of the main contributing factors for gate vibrations is the impingement of high velocity flow by shifting control point alternately from the gate upstream bottom edge to the downstream bottom edge. Other contributing factors include pressure fluctuations in the low-pressure zones at gate bottom and downstream of the gate, excessive variation of hydrodynamic forces for small vertical movements of the gate and shear flow under the gate (Sagar, 1977). As can be expected, gates with flat bottoms exhibit significant potential for vibrations due to control point shifting, as well as large down pull variations for small vertical travel. Vibrations can be significantly reduced in the upstream sealing gates with divergent flows with a lip angle of 25 deg, when the issuing water jet normally can be expected to clear the downstream bottom edge of the gate beam. It is noteworthy to mention that Japanese standards permit 20 deg. lip angle for such gates with upstream skin plate and upstream seals, as the jet flare in the upward direction seldom exceeds this angle.

Main causes of vibration are:

Separation of flow, shifting of separation point, turbulence beneath the gate bottom, inadequate aeration, air entrapment down-stream of gate, variable component of hydrodynamic forces.

Remedy:

- Proper design of gate bottom lip,
- Proper Design of spillway, pier and gate slots.
- Removal of debris/trash before gate opening.
- Adequate aeration and air entrainment device
- Leak proof ness of the gate by proper design of gate seals.
- Adopting a hydraulic hoist arrangement.
- Providing 'Snubbers' of rubber (neo-prene) material in the direction of flow.
- Keep large angle between the upstream gate face (in radial gates) and the spillway.
- Two music-note seals should be given on the top of the vertical gate when being used in tunnel spillway in order to prevent vibration caused by cavitation from leakage at the top of conduit when the gate is opened.

d. Cavitation

Example for Cavitation Failure:

1) It was considered that in1995 failure of the spillway radial gate at Folsom Dam (USA), vibration on gate have also played significantly in its failure addition to excessive trunnion pin friction. Vibrations have occurred with a gate opening of 0.73 meters and a head on the gate of 12.2 meters. Stiffening of the remaining gates was increased significantly and the replacement gate was designed to be much stiffer than the failed gate. Field tests conducted in 1998 showed no tendency for vibration of the stiffened gates

2) The diversion tunnel at the Tarbela Dam in Pakistan (Kenn and Garrod 1981) collapsed due to cavitation when the control gate became stuck during the construction phase of the dam in 1974.

3) Cavitation is the formation of vapor cavities in a liquid. Cavitation occurs in high velocity flow, where the water pressure is reduced locally because of an irregularity in the flow surface. As the vapor cavities move into a zone of higher pressure, they collapse with energy, sending out high pressure shock waves likely to damage / eroded the gate / liner steel surfaces in contact or even cause to induce vibration on gate.

4) In principle cavitation danger exists in any situation where there is flow separation without sufficient air supply. Significant or excessive cavitation damage are generally observed in high head gate on the gate members at the bottom and to the conduit downstream of the gate and gate slot areas. To estimate cavitation potential, the incipient cavitation numbers σ_i of the critical points can be determined. The maximum potential of cavitation occurs on a gate with flat bottom (e/d = 0) as flow jet springs from the upstream bottom edge and frequently reattaches to the gate bottom near the downstream bottom edge, thus creating a low pressure zone under the gate without access to aeration. Increasing e/d ratio or the lip angle or the introduction of air reduces the cavitation damage potential that occurs from collapsing vapor cavities. If the flow is not naturally aerated, measures can be taken to introduce air into the flow at critical locations along a spillway.

5) In the gates with upstream skin plates and upstream seals (U/S gates) where the flow under the gate is divergent, the increase in lip angle prevents the reattachment of jet to the gate bottom member and the cavitation potential is reduced as more and more access becomes available for aeration into the low pressure zones, under free discharge conditions. In the case of gates with downstream skin plate and downstream sealing gates where the flow under the gate is convergent, increasing e/d ratio creates a stagnation zone under the gate with positive pressure thus eliminating cavitation potential. Based upon the available data, the minimum values of e/d ratios or lip angles for diverging flow (u/s gates) as ± 20 deg and for converging flows (d/s gates) as ± 45 deg. can be preferred for minimizing cavitation potential to a reasonable degree. However prior to manufacture / installation , hydraulic model studies of the gate with associated civil structure may be conducted to identify the potential presence of cavitation and required mitigation in eliminating / reducing these undesirable hydraulic conditions e.g.by modification in gate geometry / gate lip, gate slot or induction of aeration.

2. Mechanical Failure

If gates are well monitored maintained and exercised, the chance of an inoperable spillway gate during normal operation or in case any emergency or occurrence of a large flood will be significantly reduced.

Most common cause of gate/hoist failure is the lack of lubrication on the moving components, or that the lubricant is so old that it no longer functions as designed. Inspections of the gates should also focus on wear or corrosion of wire ropes and chains and connections of the ropes and chains to the gates and the hoists. Sometime over greasing of parts / bearing may also cause to blow out of seals in turn loss of grease/ lubricant.

Hydraulic cylinders are now generally being used for more reliable operation of gates but these may also incur problems in a long run due to a various reasons. The main reasons for failure of Hydraulic cylinders are Seal Leakage due to incorrect fitting, inappropriate clearances, marking on seal grooves or corrosion etc., Fluid contamination due to abrasive particles causes damage to piston rod and seal, faulty wiper seal, damaged Rod Bearings or Piston Rods due to improper alignment between load / lifting bracket and the cylinder results in a bending or side loading. Corrosion of cylinder shell due to contaminated fluid inside a cylinder or ingress of water, broken of eye bearing due to excessive loading or any sudden impact from high pressure , failure of seals due to chemical attack cause by non-compatibility of fluid / oil with seal material or at extreme temperature (low or high) or pressure condition / side loading etc.

Regular equipment inspection and good preventive maintenance plan will decrease the chances of cylindrical failure. However it is recommended that original manufacturer of cylinder shall be referred in case of any failure and remedial action.

Other common reasons for mechanical failure of gate operation are corrosion, wear and tear of gate and hoist component like wearing of brake shoe lining, corrosion on gate / hoist steel parts & at welded or bolted joints, pitting / corrosion on drum / drum grooves etc. and due to trunnion pin friction, inadequate size and heating of motor, noise and vibration from motor and gear drives, loose / damage of connection, damaging of insulation on motor / brakes and other electrical parts , level and condition of oil in gear box / hoist component , condition of all bearings used in hoist parts, lack of lubrication at tooth contact surface, etc. Use of oversized motor than required can damage to the downstream power train. Use of geared couplings can help to reduce unwanted loads on the equipment due to misalignment but they do need lubrication. Sound and smell are also a good indicator of potential problems during hoist monitoring operations. During normal operations, the only sound should be the quiet hum of the brake solenoid. Any noise from the gears or bearings or smell of overheating components e.g. brake, motor or gear is unacceptable. Any vibrations while the hoist is running should also be noted. It is recommended that the brake, gear boxes and motor should be sent to a qualified shop / vendor for controlled disassembly, inspection and proper rectification.

Exercising of the gates will verify that the gate can travel freely within the gate bay and free from any vibration & noise at least for smaller gate openings. The walls, guide / roller tracks , piers in the area of the gate and wall plates should also be inspected for plumbness and required surface roughness for smooth operation of gate.

3. Electrical Failure

Problems with Electrical control equipment can usually be due to improper application, faulty installation or an inadequate design, inspection and maintenance of electrical system.

Most common failure of gate in Electrical Works is due to failure of Main Power Supply for gate operation, These are mainly due to tripping of MCB, defective control transformer, damaging of incoming feeder / cables / push button or contact at control panels, selection of key in local / remote panel, overload, limit switch, Interlocking contact between the terminals / contact, low voltage supply at contacts etc.

Limit switches are other of the most vulnerable components of the gate hoist. Since limit switches control over hoisting and hence its failure can have catastrophic consequences. Other problems are due to icing or corrosion on electrical contacts causes to failing in its actuation, failure of spring, breaking of limit switch arms and loss of calibration etc. These switches should be reliable and suitable for functioning with subjected loads.

Failure in Motor are generally attributed to absence of power supply, failure of motor starter and MCB''S, low voltage, burning of brake coil etc. Motor starter protects and regulates the performance of the motor and nonworking of starter may be due to no incoming supply , engagement of limit switches, defective contacts, incorrect circuit wiring , abrasive or non-conducting cement / severe dust environment that clogged the starter causing the overload relay thus power supply to trip etc. Starters should be provided with anti-vibration pad. Tripping of the overload relay is also due to low voltage or high draw in current and this also cause to freeze the tips of the contacts. Improper contacts due to chatter , overheating , noisy sound due to overheating , freezing etc. or loose connections at power & control connection ,likely to cause trouble at any time should always be checked periodically and got corrected / replaced along with the main line connections.

Other some problems attributed to electrical reasons are inadequate size of heater for site temperature, short circuit, dirty / dusty environment, wrong setting of overload relay, broken parts, wear of contact surface, low pressure of contact, and defect in spring of contact etc.

Amperage readings should be taken on a regular basis which can provide a history of the changes in the hoist and gate system. Regular tracking of Power consumption by power meter and Mul-timeter for Voltage / Current can provide instantaneous information in hoist inspection /monitoring and history of health of hoists.

Adequacy in Design and Engineering

Equipment under hydro mechanical works (gates and hoist), associated embedment and structures should be adequately designed for safe, functional and reliable operation for the all subjected / expected loads and as per standard with best adopted practices.

Following minimum loads / criteria shall be considered in the design of gate structure and all associate equipment.

- Full hydrostatic load on water side of gate corresponding to normal design head.
- Full hydrostatic load on water side of gate corresponding to emergency water head.
- Hydrostatic, frictional forces, hoist loads and hydro-dynamic force during gate operation (including partial opening as applies) with any water level from gate sill.
- Silt load on specified gate as per design data.
- Earthquake Loads (coefficient of horizontal earth quake acceleration and vertical earthquake acceleration shall be taken as per project site).
- Dead weight of the gate/structures.
- Wind Load as per IS: 800 / IS: 875.
- The design of fixed hoists /gantry crane and associated component /structures shall be designed as per relevant standards and technical specifications for the given design data. The design pressure for the hydraulic cylinder shall be taken as 20 N/mm^2.
- Any other subjected loads e.g. impact load, temperature load, breakdown condition of motor, Lifting force etc. or recommended by the manufacturer / other standards.
- The hoist capacity of Gantry crane or fixed hoists shall be computed considering all weights like dead weight of gate/stoplog unit, dead weight of lifting beam, other attachments/counter weight as applicable, all opposing forces including friction, pre-compression of seal, differential/unbalanced head on gate unit, force due to buoyancy/ hydrodynamic, up thrust / down pull on seals/gate parts etc. as required.
- In the calculation of hoist capacity, starting coefficient of friction shall be considered. A minimum reserve of 20% over and above the computed value shall be provided however it shall not be less than minimum specified value as per design data or for consideration for wearing on seals co.
- Pre stressed anchorage arrangement should be preferred than convention anchorages specifically for large Radial gates (Load) as non re stressed system (conventional anchorages) allow tension & greater deflections of the trunnion girder likely to cause structural cracking in concrete (bonded anchors) and require a large cross-sectional area of steel (embedded large-diameter rods or built-up sections).
- Gated spillway bays should always be constructed with provision of stop logs on upstream of main spillway gate. Stop logs are generally designed for maintenance purposes and operated under balanced head only. Stop log or Bulkheads may also be designed for unbalance/ flow-

ing water condition) to avoid losses of water or other consequences through uncontrolled spillway or in case spillway gate is stuck and in non-operative condition.

- Gates for surface spillways should be provided with adequate freeboard between the top of gates and the maximum normal reservoir level preferably to withstand for maximum Probable Maximum Flood.

S. No	Description	Standard
1	Recommendation of structural design of fixed wheel gates.	IS: 4622
2	Recommendation of structural design of Radial gates.	IS: 4623
3	Recommendations for structural design criteria for low head slide gates.	IS: 5620
4	Recommendation for structural design of medium & high head slide gates	IS: 9349
5	Code of practice for design of rope drum and chain hoists for hydraulic gates.	IS: 6938
6	Code of practice for electric overhead Traveling cranes and gantry cranes other than steel work cranes.	IS: 3177-1999
7	Code of practice for design, manufacture, erection & testing of cranes & hoists.	IS: 807-1976
8	Code of practice for use of structural steel in general building construction.	IS: 800
9	Design criteria of hydraulic hoists for Gates.	
9 (a)	Hoist capacity, Design of hoist components Like hydraulic cylinder, cylinder head, stem, Piston etc.	IS: 10210
9 (b)	Hydraulic operating system (Power Pack) & Its components like oil tank, pump and motor, Valves, piping etc.	DIN:19704 (part II)
10	Code for unfired pressure vessels	IS: 2825
11	Code of practice for structural safety of buildings loading Standard.	IS: 875

Table E- 1List of Standards for the Design of Hydro mechanical Equipment

Minimum guidelines for the basic design of equipment are drawn up in above criteria. In addition to this, Design shall meet the technical specifications and relevant standard of latest edition and shall ensure the quality, advance technology, optimum and safe in operation. In closed position, the gate must be watertight and sealing must be reliable. Other best practices and relevant standard for the calculations of forces /stresses shall be adopted if the same is not covered by the Indian Standard and technical specification. The structure shall also have adequate rigidity, appropriate bottom / surface profile, sealing and other arrangement to provide stable and smooth operation with good hydraulic performance so as to avoid any adverse effect on structure e.g. by vibration, cavitation, erosion /abrasion etc. Adequate clearances between the structures and from concrete surfaces shall be provided. Adequate and safe aeration provision on downstream /behind of gate as necessary should be provided. The variety and specification of parts and component shall be reasonably minimized and standardized.

Detailed drawings (shop / construction drawings) should be prepared conforming to Design and arrangement and having clear, correct and adequate details for fabrication and installation of equipment. The design, general arrangement and engineering of equipment to be adopted should have well experience base and tested for application and condition. The design and engineering of component should be carried out from well qualified and experience agency / personal of similar works for the intended objectives.

The given arrangement and engineering shall also support appropriate QA/QC requirement, fabrication, welding, fatigue / fracture analysis and control, transportation, erection, operation, inspection, handling, maintenance, repair and servicing during and post construction/operation period. Design of system and type of component to be adopted should have readily market availability for long duration even for life of system / project.

Probable Reasons for Design failure:

In most cases design deficiencies are due to failure to recognize loading possibilities for which the gates and their structures should have been designed.

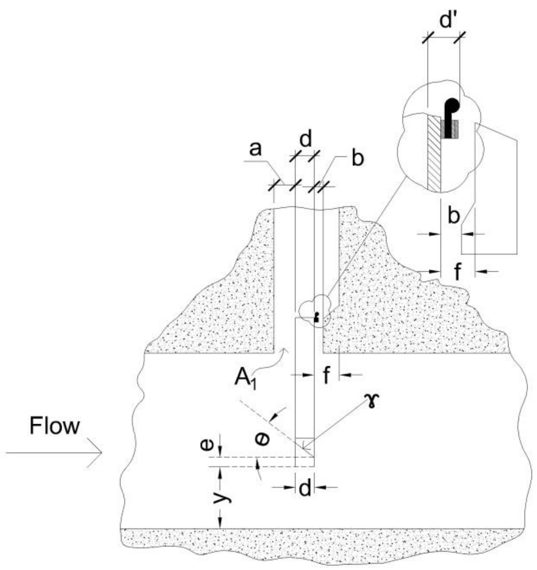


Figure E- 2 Vertical Gate in Tunnel/outlet with Downstream Skin Plate with Downstream seal

1. Hydrodynamic Load

In absence of any flow i.e. closed condition of gate, gate rests under hydrostatic balance and vertical component of hydraulic forces on gate are due to buoyancy only. During gate opening or gate in partially open condition, this hydrostatic balance is broken due to separation near the gate / gate bottom and gate /conduit operating with high velocities reduces the local pressure leads to development of hydrodynamic forces. These hydrodynamic forces are not present in Vertical gate with upstream sealing or in Radial gate as flow of water follow the same direction of movement as the gate. While in Vertical gate with downstream sealing, direction of flow is not parallel / same with the direction of gate movement creates hydrodynamic pressure (Down pull) in conduit. Various mathematical analysis have been studied for estimating the hydrodynamic forces (uplift and down pull) which may be adopted during design of gates without model studies. However Model study is more reliable for correct determination of such forces.

2. Down pull

Down pull is the net downward hydraulic force on the gate under hydrodynamic conditions. To minimize down pull so as to reduce the hoist capacity can be the primary objective of many gate designers. Factors affecting the down pull includes: a reacting head upstream of gate, on gate cross section are, Gate opening (y), gate bottom shape, gate lip (e/d ratio), slope (θ)/ curvature of bottom shape/edge, upstream clearance at gate shaft (a), downstream clearance (b) downstream seal projection (d), recess on gate shaft (f) , see Figure E-2.

With increasing lip angle (e/d ratio) increases the pressure and reduces the velocity near bottom thus reduces magnitude of down pull, cavitation and vibration but it can also create sufficient uplift which can prevent the gate closure with upstream sealing gates. Minimizing the down pull should not be the primary objective of a gate designer, provided the gate is otherwise satisfactory with regard to cavitation and vibration. Down pull if skillfully developed serves as a very useful tool for the gate designer to ensure self-closure of the gates thus provision of conservative hoist capacity is advisable than preference to marginal increase in cost of hoist.

For primary estimation purposes, Lip angle of about 25 deg. or e/d ratio of 0.45 is considered adequate to avoid net uplift on the upstream sealing gate. For downstream sealing gate, a lip angle of 45deg., with a sloping bottom plate, is preferable to minimize down pull as such slope ensures positive water pressures on the sloping plate at all gate positions, which plays a desirable role to reduce vibration potential. Additionally increase in upstream clearance at gate shaft and higher downstream seal projection may increases the down pull forces whereas provision of recess in downstream wall is effective in reducing down pull forces.

3. Uplift

Local uplift forces can exist under hydrodynamic conditions on the gate bottom. In fact, it is desirable that positive pressures exist on the gate bottom to prevent cavitation potential. A 45 deg lip with a sloping bottom plate in downstream sealing gates with convergent flow under the gate is proposed because such local uplift or positive pressures exist throughout gate bottom. However, "net uplift" on the gate which is caused by local upward forces on the gate bottom exceeding the downward force on the gate is not desirable; as such "net uplift" forces can prevent the gate closure or even to gate catapulting. A gate designer should encourage the local upward positive pressures along the gate bottom to avoid cavitation but must also be careful to avoid "net uplift" forces on the gate.

4. Air Requirement

Air Vents is required for installation of gates with downstream skin plate and seals. There may not be any need for upstream sealing gates as air demand can be provided from the gate shaft. Air supply prevents dropping down of the pressure to vapor pressure during closure / partial opening of gate and thus also reducing the chances of cavitation and vibration. This also permits drainage of the conduits and allows air to escape during filling of conduit/ tunnel.

Factors affecting air demand are gate opening, type of flow, velocity, conduit profile, bottom shape of the gate, head loss in the air vent. The location and sizing of air vents is critical for minimizing cavitation and vibration problems. The air vent should be located as near as possible to the gate, at a distance not over 2 m. Vent inlet should be located sufficiently above the maximum reservoir water level and be straight as possible with minimum abrupt changes in section i.e. bends , sharp corners , elbow etc. Generally the maximum air demand may be occur at a gate opening of about 80%.

As per Indian Standard IS: 12804 – Criteria for Estimation of Aeration Demand for Spillway, volume of air required can be estimated from the following expressions for purposes of preliminary sizing of air vents for regulating gates.

$$\frac{Q_i}{Q_w} = 0.03 * (Fr - 1)^{1.06}$$

$$\boldsymbol{Fr} = \frac{\boldsymbol{2H}^{\mathbf{0.5}}}{\boldsymbol{hc}} \text{ and } \boldsymbol{hc} = \boldsymbol{K} * \boldsymbol{h}$$

Where:

Q_a = Volume of air required in m^3/s;
Q_w = Outlet discharge in m^3/s;
Fr = Froude No. at vena contract i.e. just downstream of the gate
H = Effective Head at Vena Contract (MWL – hc)
hc = depth of water at vena contract
K = Gate contraction coefficient

For calculating Froude number, velocity is estimated based on the head corresponding to maximum reservoir level neglecting losses. The depth is estimated based on gate contraction coefficient of 0.80 for 45° lip and 0.60 for a sharp-edged gate lip (see Figures E-3 and E-4).

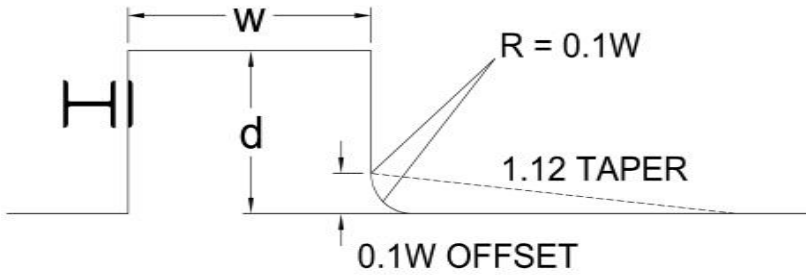


Figure E- 3 Gate Groove

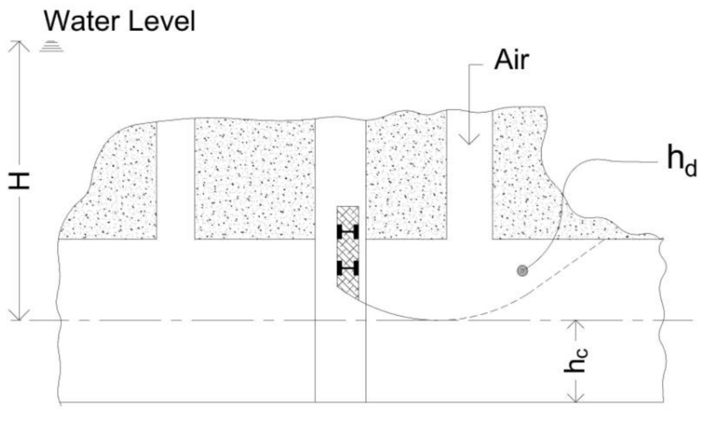


Figure E- 4. Air Vent requirement on downstream of gate in Sluices/Tunnel

For determining air vent size, the air velocity inside the pipe can be assumed as 40 m/s. Model studies can also be used to determine the requirement of air.

5. Gate Groove

Vertical-lift gates of the roller or slide type require recessed groove in abutments or piers for the movement of the gate guide rollers or slides. The flow of water across the slots causes flow separation at the upstream edge of the groove and reattachment on the downstream side. Formation of eddies and vortices are set up in the groove and the flow past the gate groove reduce pressure on the conduit results in occurrence of cavitation under high velocity. Flow conditions due to gate slots are influenced by the upstream and downstream edge shape and the cavity depth to length ratio (Figure E-2). Radiusing the upstream edge increases the flow into the cavity hence increasing energy dissipation, so this should be avoided. A radius on the downstream edge reduces energy dissipation. Between D/W ratios of 0.2 and 0.8 circulation is unstable, with periodic disturbances influencing the main flow. Cavitation of gate groove investigated by "Ball and Galperin." may be referred for reviews of cavitation in hydraulic structures. The low pressure conditions on the downstream edge of the gate groove can be improved to some degree by offsetting the downstream edge of the groove and returning gradually to the original conduit wall alignment. Sharp downstream corners of gate groove should always be offset away from the flow. The offset of the downstream corner of a gate groove should be small and related to the groove width. Within reasonable limits, this offset is not critical. Abrupt offsets into the flow and irregularities in flow surfaces are particularly troublesome. Offsets of less than 3mm will cause damage and it is important to provide smooth continuous surfaces downstream from gates operating under high heads.

6. Redundancy

Redundancy is used in critical design areas to achieve efficient and reliable operation or remedial measures during failure of any component / equipment with little / no intervention. Operation of gate should not suffer due to failure of one equipment / component, specifically for electrically control system which may suddenly fail without any prior detection. Redundant provision with suitable protections and back up means preferably to all critical control system should be used. System / component should be provided with redundant and identical arrangement so the operation can be switchover to similar alternate arrangement quickly. System should also be provided with necessary protection e, g, interlocks / bypasses, fuses /relays, etc. for any undesirable operation and prevent any malfunction or accidents.

Commercial power is typically backed up by standby generators. But diesel engine driving a generator will start and run for few hours may not be sufficient. Portable standby drive units, connected to the hoist of a gate, a mechanical flexible drive by a small diesel engine, hydrostatic oil hydraulic transmission

(Where access to the hoist is difficult), Battery driven DC motors are other alternative arrangements for power back up. Alternatively, two independent external commercial grid supplies may be provided which may then be connected to two physically segregated and protected feeder systems that provide power to the gate in addition to standby power back up / generators. Each gate can have two physically segregated switchboards, each of which can be configured with two feeders.

The gates can be operated either locally or from a central control room via a remote/Auto control system. The control system may also have some degree of redundancy even with hot redundancy of system controller as failure of one controller does not affect the loss of signal / data and auto operation of gates. The control system should also incorporate a number of interlocks for equipment protection.

Gate with hydraulic cylinders used for their operation, double end operation is the preferred mode of closure. Power pack and local control of each gate can also be made to operate Hydraulic cylinders of adjacent gate(s). Two motor driven oil pumps (one as stand by) should be provided for the operating system to ensure the operation of each gate or valve, in case one motor-pump unit fails. The portable diesel / gasoline operated mobile power pack should be kept for gate hoist operation in case of power supply failure.

7. Gate Automation

Controlling and automation of various processes is a fast growing business nowadays and automatic controlling of gate of Dam is also considered effective for its efficient operation. Automatic, monitoring and control / operation of gates is recommended for spillway in barrages/dams with many/multiple gates to regulate high discharges or prone /likely to flood / uncontrolled release or maintain the reservoir level for water / any losses or dam associated with Power stations.The primary purpose of an automatic control system or automation system is to allow through computerized control the automatic starting, stopping, safe operation, and protection of any or all gates and associated equipment being controlled. Generally the gates have been monitored and controlled by human operators for many years, both locally and remotely. Unfortunately, efficiency of operation can be affected by manual capability, rapid change of variables, lack of monitoring and availability of man / equipment. However, a computer system has the capability to effective monitoring and analysing of parameter and give optimum performance settings for opening / closing of gate in sequence for discharge regulation or quick release of discharge in event of flood and also enabling of various alarms and warning.

The automatic operation of spillway gates shall be microprocessor based consist of PLC (programmable logic controller) or RTU (remote terminal unit), PC based data server, operator stations PC and HMI (human machine interface), conventional panel boards for manual control and SCADA (Supervisory Control and Data Acquisition) software. The gate control system of the dam uses PLC to control the opening and closing of gate according to the detected level of water in a dam. The inputs of programmable logic control (PLC) device are connected with water level sensors. Relays are connected between PLC and hoist motor. The gate hoist motor is connected with relay driver which strongly resembles a schematic diagram of relay logic. The actuation of gates is implemented by PLCs as per program operation sequence and control the opening and closing of the gate(s) as and when needed.

Primary objective of gate automation system for Control and operation of Spillway Gates includes Gate position indication and monitoring of Dam / Spillway Gates, Water level indication and monitoring along with necessary alarms and warning, Monitoring and indication of discharge measurements for through Dam /Spillway Gates and differential pressure measurement and indication across Trash racks.PLC can also be a vital part of an Emergency Action Plan (EAP).

The threshold events as identified in a dam owner's emergency action plan can be included in the program and actions can be set automatically which could include not only operation of gates but also provide automated notification message / warns via redundant methods to the dam owner or local residents or other emergency stake holders/administrator.

The spillway operation is based on the strategy of balancing the reservoir inflow, available reservoir capacity and outflow, by checking and comparing measurements taken at intervals of 15 minutes. These parameters can be computed from (reservoir level, gate position and spillway discharge) by gate automation arrangement. The control panel and computing system for the spillway can calculate the actual spillway discharge depending from reservoir water level and gate opening. The control panel and computing system is interconnected with the central computer in the Dam control room (DCR) from which the command for the necessary discharge on the spillway could be provided. All signals can be sent and received to/from the DCR.

The system is to be designed for proper monitoring of gate operation and the particular gate(s) is to be cancelled from the group of gates available for selection in the event of a malfunction / non operation of any gate(s).

The discharge rate per gate is to be computed by electronic means. Water level measurement and measurement of the gate position serve as input variables. The necessary discharge rating curves should be prepared. The discharge rate from all the spillway gates shall be added and be displayed digitally as total water discharge rate and recorded.

The operator stations will be able to automatically regulate controlled opening of the gates to control outflow of water to maintain the required reservoir level as required, change the parameters of response of the gates to the inflow levels by changing the program on line without disturbing the existing parameters, read and transfer / receives the position of each gate, operate any or all the gates from any mode manual/ auto/remote, obtain audio/visual alarms and indications of various faults in the hydraulic / hoist system including real time display of gate motion of selected gate, any other communication need between DCR and other Stations if any , print outs at periodical intervals regarding the water inflow, outflow, gate openings and fault indications.

A main selector switch (key operated) for the different modes of operation (local/remote and automatic) should be provided in control cabinet (local and Remote).

All equipment should be checked for sensitivity and correct functioning and "dry" tested at site by simulating various reservoir elevations at the level-sensing equipment. All software should be tested before downloading or installing.

Local control panel should be independent and can be operated without the controllers and /or gate automated system.

Use redundant power supplies and/or use the DC battery power and a UPS (uninterruptible power supply) as an emergency backup. Use redundant controllers for critical control & communications. Redundancy for LAN and I/O network (for remote I/O drops) should be provided. All ancillary equipment should automatically go to a safe state on failure of a PLC or failure of critical instrumentation. Use a firewall /adequate security system to prevent intrusion attempts or unauthorized access.

8. Model study

The gates should be designed considering the influences of hydraulic conditions and able to operate with good hydraulic performance without cavitation and vibration. Model studies for design of gates are seldom performed but it is recommended for submerged gates / tunnel gates operating under high head / velocity. The objectives of a model study are determine the discharge characteristics of a radial gate throughout the range of gate openings, estimation of hoisting / closing capacity for gate and hydraulic down pull forces acting on a vertical-lift gate in a tunnel, requirement of air or to studying the interaction between a service gate and an emergency gate in a conduit. The model may also be used to measure pressure variations in spillway, shape of water passage / gate slot to avoid low pressure zone or sub atmospheric pressures in vicinity of gates or on the upper part of the spillway/ crest. Model test of a gate is the only reliable method for determining of hoisting / closing capacity or hydrodynamic forces e.g.down pull or uplift forces on gate which may cause non closure or catapulting of gates, specifically for gates used for emergency events such as power failure, etc.

Model study can also be useful to investigate the existing or potential problems of gate vibration or cavitation. Model studies of gates and gate installations are project specific and carried out to obtain project specific numerical results. These have limited validity and only show certain relationships of observed hydraulic parameters within the range of the experiments undertaken. Model study for the required objective should be carried out in hydraulic laboratories having well experienced of similar investigations. The hydraulic model for such investigation around gates is preferred to be built with geometrical scale in the range of 1:20 to 1:30.

Fabrication

Fabrication / Manufacture of gate , embedded parts and associated steel support structure consist basically of steel work and machining services which involves shop / layout drawing, procurement and storing of raw material, bought out items, marking, cutting, bending, welding, finishing, pre-assembly, machining, fit-up / assembly, anti-corrosive protection inspection. Use of all material and parts/equipment in fabrication /manufacture of equipment shall be as per approved drawings with good and tested quality as per relevant standards / approved reputed make and free from any defect. All welding and brazing shall be carried out relevant standard as per approved QA Plan by the qualified welders and in accordance with qualified welding procedure for the given weld type. Machining of steel surfaces shall be as per drawings and standards in order to obtain required surface finish / tolerances for the intended objective/ function. Pre-assembly / Mechanical fit up of parts shall be performed for overall verification of dimensions, fit-up , positioning, alignment etc. Field assembly mark shall be placed in this phase. Finally Anti corrosive protection including surface preparation & painting shall be carried out to steel surfaces as per approved painting schedule / technical specification conforming to Indian Standard IS -14177 / relevant Swedish standard .

Erection

A method statement / manual for the erection of equipment at site should be prepared and accordingly carried out as per instructions, standards and procedures as per method statement /manual. Erection Manual should cover the minimum information as list of drawings and other reference documents , erection sequence, instructions for handling and indication of the suspension points of large parts ,dimensions and weight of the main parts , instructions for placing, Alignment, leveling and fixation of parts ,marks for assembly , instructions for concreting of embedded parts, field joints and welds to execute (including the recommended type of electrodes) , required equipment for the erection (cranes, hoists. jacks, turn buckles etc.) It may note that erection of equipment should not include any field machining or stress relieving. The field erection of the gate components usually follows the sequence: embedded pars, gate and hoist.

Inspection

To ensure reliable operation of hydro mechanical equipment, it is essential that appropriate inspection programs should be prepared during fabrication, installation and operation.

The detailed Quality Plans for inspection and testing during manufacturing (MQAP) and field Installation (FQAP) shall be prepared and finalized before start of work. The same specification and quality system, plan and procedures shall be strictly followed by the contractor or his subcontractor or any supplier during the various stages of procurement of materials, manufacture, installation and testing etc. All testing including dimensional, functional, operational etc. should be as per approved inspection; test & quality plan (ITP /QAP) and recorded. All equipment and instruments to be used in fabrication, erection or operation shall be calibrated before the commencement of tests and at regular intervals.

An appropriate level of inspection should be carried out at manufacturer's facilities to ensure that equipment, as delivered to site conforming to specification, drawings and approved MQAP.

During installation and commissioning, inspection and testing should be performed as per FQAP to ensure that the equipment is ready to be put into service. Subsequently, there should be routine inspection and testing as per operation and manual and manufacturer recommendation of equipment to ensure that the equipment remains in satisfactory functional condition for reliable operation

1. Inspection and testing during Fabrication / Manufacturing

During manufacture, close inspection and testing should be carried out as per relevant Indian / applicable standard to ensure fabrication / manufacturer of equipment as per specification, design and drawings. Complete inspection for material and equipment including casting and forging, position, surface finish and alignment of equipment, welding procedure and quality, shop assembly, cleaning and painting etc. and conforming to MQP should be done. Also ensure that fabrication / manufacture of steel structure (embedded parts , gates and other associated structure) should meet the tolerances as per relevant Indian / applicable Standard. All raw material and bought out items / components like steel plates, forging, casting , rubber seals, fasteners, motor, pump, gear box, bearings, brakes, electrical component including cables, etc, should be of reputed approved make and verified from manufacturer test certificates. Complete Inspection for the hoists for material, casting and forging , hoist unit (Drive unit comprising motor, gear box and brake), base frame, Hoist drum, open reduction line shaft, bearings, gate position indicator, hoist rope and fixture, control panels, manual gate operation lubrication of gear and bearing, oils, assembly of hoist, anti-corrosive protection / painting to structure and component etc. shall be made as per relevant Indian / applicable standard at the place of manufacture prior to dispatch .

Following Indian code may be referred for inspection and testing during manufacture.

a) IS:7718 – Recommendations for Inspection, Testing and Maintenance of Fixed wheel and Slide gates
b) IS:10096 (P-1/Sec.1) – Recommendations for Inspection, Testing and Maintenance of Radial gates and their Hoists: For Gates
c) IS:10096 (P-1/Sec.2) – Recommendations for Inspection, Testing and Maintenance of Radial gates and their Hoists: For Rope drum hoists

2. Inspection during Installation / Commissioning

During installation, close inspection should be carried out to ensure good alignment of equipment. Gate parts and equipment received at site have been manufactured as per drawing, supplied with match markings for assembly and erection, required surface protection and without any damage. Critical dimensions including for concrete block out shall be checked and corrected before erection. All references / levels as per civil structure shall be established to ensure erection of equipment at proper locations. Care shall be taken for proper storage and handling of the components at site and during erection.

In particular, straight edge checks and overall survey checks should be performed on critical embedded parts, such as sill and lintel beams, sealing faces and roller paths of gates. Precise and latest survey techniques for positioning and alignment may be used. Gate rollers are designed to permit fine adjustment in the field, so contact between rollers and roller path should be checked. Alignment of hoist equipment, line shafts, etc. should be checked to ensure these are consistent with the couplings used and the operation of the hoists. During erection and commissioning, records of all inspections and tests should be maintained. Wiring and merger tests of all electrical systems with phase and rotation of all equipment should be checked.

Functional testing of all equipment should be performed in the dry conditions. Sealing of gate seals and valve seals can be examined visually, and adjustments made. Oil levels in gear boxes and lubrication at all lube points should be checked. Settings and function of all limit and alarm switches should be tested. Clearances of all moving parts, speed of operation, seating of any wire ropes on drums and sheaves, current drawn by motors, speed of operation of opening and closing of valves or gates, and operation of solenoids and fan in brakes should be checked. Particular care should be taken to ensure that air intake to fan brake is clear, and cannot be blocked, since this could lead to overheating and a runaway condition of the brake.

On completion of dry testing, wet testing of equipment should be performed, preferably under a range of operational conditions. Smoothness of operation, seal leakage, operational speeds, and currents drawn can all be checked and any problems observed can be addressed and corrected.

Following Indian code shall be referred for detailed inspection of block out, embedded parts,

gates, hoists and their testing during installation.

a) IS:7718 – Recommendations for Inspection, Testing and Maintenance of Fixed wheel and Slide gates

b) IS:10096 (P-1/Sec.3) – Recommendations for Inspection, Testing and Maintenance of Radial gates and their Hoists: For Gates and Rope Drum Hoists

3. Inspection and Maintenance during operation (after Erection and Commissioning)

In order to ensure reliability of gate operation, periodical inspection followed by required maintenance of the installed gates with all other associated equipment shall be carried out regularly .A program for routine inspection and maintenance should be set up, and records kept of all inspections. Inspection and maintenance may be suitably broken down into different levels, such as 'One-Month', '6-Month' and 'Annual' inspections. The intensity of inspection and maintenance at each stage obviously depends on the facility and its importance, but typically a 'one-month' inspection would involve checking the operational condition of equipment, including controls, brakes, oil levels, and any signs of rusting or accumulation of debris which might jeopardize proper functioning.

A '6-month' inspection would include all of the above, but might also include complete opening and closing of all gates and valves to check functioning over the operational range. Operation of heating and bubbler systems would be checked if provided as per site condition and this would be scheduled to occur before and after the winter season. Lubrication would be carried out as required.

On an 'annual' basis, thorough operating tests should be performed, and these would normally coincide with the '6- month' inspection above. Gate or valve operation from fully open to fully closed position, and speed of operation would be checked. Settings of limit switches (slack rope, rotating cam, etc.) would be recorded, as would all local and remote indicators of gate or valve position. Checks would be performed for wear of equipment such as seals and brake linings. Any necessary adjustments and lubrication should be performed.

In cases where there is known to be possibility of some deterioration, such as cavitation around guides of high head gates or fatigue in the vanes of hollow cone valves, inspection should be carried out and if required, plans made for restriction of operations until any necessary maintenance has been performed.

Essential spare parts and tools must be kept available at site for immediate maintenance, repair and replacement. Condition of spares should be checked periodically, properly stored, Apply painting, Lubrication /coating as needed before use. Calibration of Tools / equipment should be reviewed and carried out periodically.

Following Indian code shall be referred for periodic and detailed inspection, testing and maintenance of embedded parts, gates, hoists after installation of gates, hoist and allied equipment.

a) IS:7718 – Recommendations for Inspection, Testing and Maintenance of Fixed wheel and Slide gates

b) IS:10096 (P-3) – Recommendations for Inspection, Testing and Maintenance of Radial gates and Rope drum hoists: After Erection

c) IS:13053–Commissioning and Maintenance of complete hydraulic systems– Recommendation

Testing of Gates (after commissioning)

- The gate shall be tested in a dry condition with hoist duly connected for its smooth working. The gate should be fully closed and fully opened and it should be ensured that there is no obstruction and no undue efforts required for its operation. If the gate is not going down by its own weight or found tight in some position, reasons should be investigated and remedied instead of forcing the gate down.

- The testing of gate seals in dry condition should be done by suitable means, such as by viewing the contact surface against a light source.

- In case of rubber seals, water should be poured over the seals so that there will not be dry friction of the seals. In case of metal to metal contacts, oil or grease is to be used. No grease or lubricant is to be used for rubber seals.

- There shall be no noise of friction or any other noise, no signs of excessive friction, no jerky performance, no dug in any position, no dangling of the gate, no twist in rubber seals and rubber seals are not over-pressed.

- The gate is to be first kept resting on sill beam that is in closed position. The leakage test can be done in this position by using a suitable pump with necessary arrangements of jetting water at 1.5 times the designed pressure on sealing positions from bottom to top. All joints, if any, shall be tested to ensure prefect working of the gate.

- The gate should be fully opened and closed to ensure full opening and satisfactory closing. The time required for 300mm opening or closing of the gate is to be recorded.

- The arrangements provided for preventing the travel of the gate or hoist beyond the designed limit are tested and checked for proper working.

- In case of hydraulic hoists, it should be ensured that oil pressures are within the designed values.

- The full load current required for movement of the gate on load shall be measured and checked against the designed value.

- When the rainy season starts, visual inspection of gates on load shall be made and they shall be lowered and raised several times to make sure that everything is in order.

- When the water starts overflowing, lower the gates to hold the water to half the height of gates. In this position the seals may be tested and any leakage shall be attended to. The gates may also be operated up and down with this water load and operation of hoist shall be observed.

- The gates shall also be tested in a similar way against full water load.

- During the testing of gate in dry condition and under water pressure, the following observations shall be made and a record be kept:
  - a) Movement of gate and indication of jamming, if any.
  - b) Effective stop is achieved by the gate stops wherever provided.
  - c) Speed of opening and closing, and the current requirement at specified voltage.
  - d) Operation of brakes and limit switches.
  - e) Efficiency of guide rollers to check the side sways of the gate.
  - f) Correctness of indication by local position indicators.
  - g) Synchronization of remote position indicators.
  - h) Vibration of gate, hoist and full civil structure.

Storage

Rubber Seals of gates should be kept in straight length and stored in flat position. The seals should not be coiled. These should be properly wrapped with jute cloth throughout its length. Preferably stacking of seals should be in wooden boxes or wooden platform. The seals should be protected from excessive heat or direct sun. The seals should not be placed near oil or grease or paint or any sharp material which may damage the seals. All type of seals must be tagged with item no., type, respective gate etc. for easy identification.

All type of electrical items like -pushbuttons, indicating lights, relays, contactors, motors etc.; should be kept in poly-bags or covered with polythene sheet or separate boxes at stacking place with their identification mark, type , quantity etc. It must ensure that these items will remain protected from water, dust, rust, humidity and any physical damages.

All bolts, nuts, fastener etc. should be kept in sealed poly-bags. Before packing, these must be cleaned and dipped in oil or apply with grease to protect rusting. Provide their identification marks, quantity, size, numbers etc.

Brakes, Couplings, Gear Boxes, Plummer blocks etc.; should be wrapped with PVC and kept in shock proof wooden pack with their identification marks, name, type, size, ratings etc.

Ropes must be stored in a well ventilated covered shed. As far as possible place should be free from moisture, dust and fumes. During the period of storage, a suitable rope lubricant should be applied every 3 months to the outer layer of the rope to protect the same from corrosion because, once started, corrosion may develop and render the rope unsuitable for use. Once in every three months the wooden reel containing rope should be rolled through 180° to prevent draining out of the lubrication from the upper layers of the rope. To protect the wooden reels from the attack of termites the floor should be cemented. In no case the reels be put on ground on uncommented floor.

Bearings should normally be applied with a rust preventive compound before packaging and they may then be stored in the original unbroken package for a number of years provided the relative humidity of the storage room does not exceed 60%.

Preparation of Operation and Maintenance Manual

For systematic operation and maintenance of the gates and their operating equipment, the availability of comprehensive Operation and Maintenance manual for the equipment is essential. These should be prepared for each hydraulic gate installation of the project/ Dam. Operation staff shall be made well conversant with project equipment &including detailed operating procedure from their controls, Details and Make of component used , Inspection and maintenance schedule , lubrication and painting requirements , list of spares and tools , Trouble s

hooting charts, storage of parts, safety requirements during inspection and maintenance, ,all reference drawings and diagrams/procedure, manufacturer catalogues etc. O&M manual shall also meet the recommendation of manufacturer of HM equipment.

APPENDIX F – GLOSSARY OF TERMS FOR DAM SAFETY

A.

Abrasion erosion. Damage caused by abrasive effects of waterborne silt, sand, gravel, rocks, ice and other debris rolling and grinding against a concrete surface. Abrasion erosion is readily recognized from the smooth, worn-appearing concrete surface.

Abutment. That part of the valley side against which the dam is constructed. An artificial abutment is sometimes constructed, as a concrete gravity section, to take the thrust of an arch dam where there is no suitable natural abutment. The left and right abutments of dams are defined with the observer viewing the dam looking in the downstream direction, unless otherwise indicated.

Adit. A nearly horizontal underground excavation in an abutment having an opening in only one end. An opening in the face of a dam for access to galleries or operating chambers.

Adverse consequences. Negative impacts that may result from the failure of a dam. The primary concerns are loss of human life, economic loss (including property damage), lifeline disruption, and environmental impact.

Aging. The process of changing properties over time.

Air release valve. A valve, usually manually operated, which is used to release air from a pipe or fitting.

Air vent: A system used to permit air to enter the conduit to prevent collapse or to prevent the formation of low pressures within flowing water that could lead to cavitation and its possible attendant damage.

Ancillary. Features in a stilling basin such as chute blocks, baffle blocks, and end sills.

Annulus: The space between an existing conduit and a newly installed slip liner.

Appurtenant structure. Ancillary features of a dam such as outlets, spillways, powerplants, tunnels, etc.

Approach channel. See Entrance Channel.

Apron. A section of concrete or riprap constructed upstream or downstream from a control structure to prevent undercutting of the structure.

Articulated concrete block (ACB). A concrete block unit when installed and interconnected with other block units forms an erosion resistant revetment with specific hydraulic characteristics. The individual units are connected by geometric interlock, cables, ropes, geotextiles, geogrids, or a combination thereof, and typically overlay a geotextile for subsoil retention.

Audit (Technical audit - TA): Audit (comprehensive technical review) performed by an auditor, engineer or subject-matter expert, to evaluate deficiencies or areas of improvement in a process, system or proposal.

Axis of dam. The vertical plane or curved surface, chosen by a designer, appearing as a line, in plan or in cross-section, to which the horizontal dimensions of the dam are referenced.

B.

Backwater curve. The longitudinal profile of the water surface in an open channel where the depth of flow has been increased by an obstruction, an increase in channel roughness, a decrease in channel width, or a flattening of the bed slope.

Baffle block. A block, usually of concrete, constructed in a channel or stilling basin to dissipate the energy of water flowing at high velocity.

Base thickness. Also referred to as base width. The maximum thickness or width of the dam measured horizontally between upstream and downstream faces and normal to the axis of the dam, but excluding projections for outlets or other appurtenant structures.

Bedrock. Any sedimentary, igneous, or metamorphic material represented as a unit in geology; being a sound and solid mass, layer, or ledge of mineral matter; and with shear wave threshold velocities greater than 2500 feet/second.

Berm. A nearly horizontal step in the sloping profile of an embankment dam. Also a step in a rock or earth cut.

Boil. A disruption of the soil surface caused by water discharging from below the surface. Eroded soil may be deposited in the form of a ring (miniature volcano) around the disruption.

Borrow area. The area from which natural materials, such as rock, gravel or soil, used for construction purposes is excavated.

Breach. An opening through a dam that allows the uncontrolled draining of a reservoir. A controlled breach is a constructed opening. An uncontrolled breach is an unintentional opening caused by discharge from the reservoir. A breach is generally associated with the partial or total failure of the dam.

Breach analysis. The determination of the uncontrolled release of water from a dam (magnitude, duration, and location), using accepted engineering practice, to evaluate downstream hazard potential. Breach inundation area – An area that would be flooded because of a dam failure.

Breaching Section. (IS-9410-Part 9). A low earth bund or dike built across a low saddle in the rim of the reservoir, or in embankment reach intended to be washed out when the water reaches a predetermined elevation without endangering any other aspects by any further rise. It is also called 'Fuse Plug Spillway'.

Breast wall. A suspended wall on top of the spillway, spanning between the piers, so as to create a rectangular opening above the crest level to pass the flow of water stored behind the wall.

Bulkhead. A partition or structure separating compartments or to hold back water.

C.

Caisson. A watertight chamber or hallow floating box used in construction work under water.

Camber. The extra height added to the crest of embankment dams to ensure that the freeboard will not be diminished by foundation settlement
or embankment consolidation. The amount of camber is different for each dam and is dependent on the amount of foundation and embankment settlement expected to occur.

Cavitation. According ACI 2000. Pitting of a material caused by implosion, that is, the collapse of vapor bubbles in flowing water that form in areas of low pressure and collapse as they enter areas of higher pressure.

Cavitation. The formation of partial vacuums in fast-flowing water caused by sub atmospheric pressures immediately downstream from an obstruction or offset. Usually accompanied by noise and vibration. The formation of voids or cavities caused in a liquid due to turbulence or temperature which causes the pressure in local zones of the liquid to fall below the vapor pressure.

Cavitation damage. Damage caused when partial vacuums formed in a liquid by a swiftly moving solid body (e.g. a propeller) pit and wear away solid surfaces (e.g. metal or concrete). The attack on surfaces caused by the implosion of bubbles of water vapor.

Channel. A general term for any natural or artificial facility for conveying water.

Chimney drain. A vertical or inclined layer of permeable material in an embankment to control drainage of the embankment fill.

Chute. The water carrier open channel used for conveying water from the reservoir to the energy dissipater or apron to meet the river channel downstream of the dam. It is an inclined conduit or open channel having a steep slope for conveying water over high drops.

Chute block. One of a series of upright obstructions placed at the upstream entrance to a stilling basin to increase the effective depth of the incoming flow, break the flow up into a number of small jets, and help create turbulence for energy dissipation.

Clearance. A procedure used to establish, under tightly controlled discipline, a safe environment for maintenance, repair, or inspection. It includes systematically isolating pertinent equipment from all sources of hazardous energy.

Cofferdam. A temporary structure enclosing all or part of the construction area that construction can proceed in the dry. A diversion cofferdam diverts a stream into a pipe, channel, tunnel, or other watercourse.

Compaction. Mechanical action that increases the density by reducing the voids in a material.

Comprehensive EAP exercise. An in-depth exercise of an EAP that involves the interaction of the dam owner with the state and local emergency management agencies in a stressful environment with time constraints. Functional and full-scale EAP exercises are considered comprehensive EAP exercises.

Concrete lift. The vertical distance between successive horizontal construction joints.

Concurrent floods. Flood flows expected at a point on the river system below a dam at the same time a flood inflow occurs above the dam.

Conduit. A closed channel to convey water through, around, or under a dam. Covered portion of spillway between the gate or crest structure and the terminal structure, where open channel flow and/or pressure flow conditions may exist. Portion of an outlet works between the intake structure and gate chamber and/or the control structure. Conduits are located beneath embankment dams and within concrete dams. Conduits are concrete lined or concrete/steel lined. A pipe, box, or horseshoe structure, or natural channel that is constructed by means of "cut and cover". A conduit can convey water or house other conduits or pipes.

Consequences. Potential loss of life or property damage downstream of a dam caused by floodwaters released at the dam or by waters released by partial or complete failure of dam. Also effects of landslides upstream of the dam on property located around the reservoir.

Construction joint. The interface between two successive placements or pours of concrete where bond, and not permanent separation, is intended.

Contact grouting. Filling, with cement grout, any voids existing at the contact of two zones of different materials, i.e., between a concrete tunnel lining and the surrounding rock.

Contraction joint. Contraction joints are joints placed in concrete to provide for volumetric shrinkage of a monolithic unit or movement between monolithic units. Contraction joints have no bond between the concrete surfaces forming the joint. Except as otherwise provided for dowels, reinforcement is never continuous across a contraction joint. Contraction joints will not transfer moment and will not transfer shear unless keyed.

Control joint. Control joints are joints placed in concrete to provide for control of initial shrinkage stresses and cracks of monolithic units. Control joints are similar to contraction joints except that reinforcement is always continuous across the joint. Control joints are unbounded joints to provide weak areas for cracking. Control joints will transfer moment, but will not transfer shear unless keyed.

Control Structure. A major component of a spillway that regulates and controls the outflows from the reservoir.

Core. A zone of low permeability material in an embankment dam. The core is sometimes referred to as central core, inclined core, puddle clay core, rolled clay core, or impervious zone.

Core wall. A wall built of relatively impervious material, usually of concrete or asphaltic concrete in the body of an embankment dam to prevent seepage.

Corrosion. Wear or dissolving away through chemical action as by rusting, or acids. The gradual decomposition or destruction of a material by chemical action, often due to an electrochemical reaction. Corrosion may be caused by stray current electrolysis, galvanic corrosion caused by dissimilar metals, or differential concentration cells. Corrosion starts at the surface of a material and moves inward.

Crack. Linear discontinuity produces by fracture. Elongated narrow opening.

Creep. A process of deformation that occurs in many materials where the load is applied over an extended period.

Crest gate (Spillway Gate). A gate on the crest of a spillway to control the discharge or reservoir water level.

Crest length (length of dam). The measured length of the dam along the crest or top of dam. This also includes the spillway, power plant, navigation lock, fish pass, etc., where these form part of the length of the dam. If detached from the dam, these structures should not be included.

Crest of dam. See top of dam.

Cross section. An elevation view of a dam formed by passing a plane through the dam perpendicular to the axis.

Cured-in-place pipe (CIPP). A hollow cylinder consisting of a fabric tube with cured (cross-linked) thermosetting resin. Interior or exterior plastic coatings, or both, may be included. The CIPP is formed within an existing conduit and takes the shape of and fits tightly to the conduit.

Cutoff trench. A foundation excavation later to be filled with impervious material to limit seepage beneath a dam.

Cutoff wall. A wall of impervious material usually of concrete, asphaltic concrete, or steel sheet piling constructed in the foundation and abutments to reduce seepage beneath and adjacent to the dam.

D.

Dam. An artificial barrier that has the ability to impound water, wastewater, or any liquid-borne material, for the purpose of storage or control of water.

Afterbay dam. See regulating dam.

Ambursen dam. A buttress dam in which the upstream part is a relatively thin flat slab usually made of reinforced concrete.

Arch dam. A concrete, masonry, or timber dam with the alignment curved upstream so as to transmit the major part of the water load to the abutments.

Buttress dam. A dam consisting of a watertight part supported at intervals on the downstream side by a series of buttresses. Buttress dam can take many forms, such as a flat slab or massive head buttress.

Concrete Face Rock-fill dam (CFRD). A rock-fill dam with a concrete slab on its upstream slope as impermeable element. See Rock-fill dam.

Crib dam. A gravity dam built up of boxes, crossed timbers or gabions, filled with earth or rock.

Diversion dam. A dam built to divert water from a waterway or stream into a different watercourse.

Double curvature arch dam. An arch dam that is curved both vertically and horizontally.

Earth dam. An embankment dam in which more than 50% of the total volume is formed of compacted earth layers are generally smaller than 3-inch size.

Embankment dam. Any dam constructed of excavated natural materials, such as both earthfill and rockfill dams, or of industrial waste materials, such as a tailings dam.

Gravity dam. A dam constructed of concrete and/or masonry, which relies on its weight and internal strength for stability.

Hollow gravity dam. A dam constructed of concrete and/or masonry on the outside but having a hollow interior and relying on its weight for stability.

Hydraulic fill dam. An earth dam constructed of materials, often dredged, which are conveyed and placed by suspension in flowing water.

Industrial waste dam. An embankment dam, usually built in stages, to create storage for the disposal of waste products from an industrial process. The waste products are conveyed as fine material suspended in water to the reservoir impounded by the embankment. The embankment may be built of conventional materials but sometimes incorporates suitable waste products.

Masonry dam. Any dam constructed mainly of stone, brick, or concrete blocks pointed with mortar. A dam having only a masonry facing should not be referred to as a masonry dam.

Mine tailings dam. An industrial waste dam in which the waste materials come from mining operations or mineral processing.

Multiple arch dam. A buttress dam comprised of a series of arches for the upstream face.

Overflow dam. A dam designed to be overtopped.

Regulating dam. A dam impounding a reservoir from which water is released to regulate the flow downstream.

Rock-fill dam. An embankment dam in which more than 50% of the total volume is comprised of compacted or dumped cobbles, boulders, rock fragments, or quarried rock generally larger than 3-inch size.

Roller compacted concrete dam (RCC). A concrete gravity dam constructed by the use of a dry mix concrete transported by conventional construction equipment and compacted by rolling, usually with vibratory rollers.

Rubble dam. A stone masonry dam in which the stones are unshaped or uncoursed.

Saddle dam (or dike). A subsidiary dam of any type constructed across a saddle or low point on the perimeter of a reservoir.

Tailings dam. See mine tailings dam.

Dam Crest Elevation / Top of the Bank Level. The lowest elevation at which water can flow over the top of the dam, not including flow through the spillway. If crest elevations for the masonry/concrete and earthen sections are different, it may be recorded accordingly.

Dam failure. Catastrophic type of failure characterized by the sudden, rapid, and uncontrolled release of impounded water or the likelihood of such an uncontrolled release. It is recognized that there are lesser degrees of failure and that any malfunction or abnormality outside the design assumptions and parameters that adversely affect a dam's primary function of impounding water is properly considered a failure. These lesser degrees of failure can progressively lead to or heighten the risk of a catastrophic failure. They are, however, normally amenable to corrective action.

Dam incident. All problems occurring at a dam that have not degraded into 'dam failure' and including the following:

a) Structural damage to the dam and appurtenant works;

b) Unusual readings of instruments in the dam;

c) Unusual seepage or leakage through the dam body;

d) Change in the seepage or leakage regime;

e) Boiling or artesian conditions noticed below an earth dam;

f) Stoppage or reduction in seepage or leakage from the foundation or body of the dam into any of the galleries, for dams with such galleries;

g) Malfunctioning or inappropriate operation of gates;

h) Occurrence of any flood, the peak of which exceeds the available flood discharge capacity or 70% of the approved design flood;

i) Occurrence of a flood, which resulted in encroachment on the available freeboard, or the adopted design freeboard;

j) Erosion in the near vicinity, up to five hundred meters, downstream of the spillway, waste weir, etc.; and

k) Any other event that prudence suggests would have a significant unfavorable impact on dam safety.

Dam Inspection. On-site examination of all components of dam and its appurtenances by one or more persons trained in this respect and includes examination of non-overflow portion, spillways, abutments, stilling basin, piers, bridge, downstream toe, drainage galleries, operation of mechanical systems (including gates and its components, drive units, cranes), interior of outlet conduits, instrumentation records and record-keeping arrangements of instruments.

Dam owner. The Central Government or a State Government or public sector undertaking or local authority or company and any or all such persons or organizations, who own, control, operate or maintain a specified dam.

Dam safety. Dam safety is the art and science of ensuring the integrity and viability of dams such that they do not present unacceptable risks to the public, property, and the environment. It requires the collective application of engineering principles and experience, and a philosophy of risk management that recognizes that a dam is a structure whose safe function is not explicitly determined by its original design and construction. It also includes all actions taken to identify or predict deficiencies and consequences related to failure, and to document, publicize, and reduce, eliminate, or remediate to the extent reasonably possible, any unacceptable risks.

Dam safety program purposes. The purposes of a dam safety program are to protect life, property, and the environment by ensuring that all dams are designed, constructed, operated, and maintained as safely and as effectively as is reasonably possible. Accomplishing these purposes requires commitments to continually inspect, evaluate, and document the design, construction, operation, maintenance, rehabilitation, and emergency preparedness of each dam and the associated public. It also requires the archiving of documents on the inspections and histories of dams and the training of personnel who inspect, evaluate, operate, and maintain them. Programs must instill an awareness of dams and the hazards that they may present in the owners, the users, the public, and the local and national decision-makers. On both local and national scales, program purposes also include periodic reporting on the degree of program implementation. Key to accomplishing these purposes is to attract, train, and retain a staff proficient in the art and science of dam design.

Densification A means of improving the strength of soil by making it denser, usually by physical compaction.

Design water level. The maximum water elevation, including the flood surcharge that a dam is designed to withstand.

Design wind. The most severe wind that is reasonably possible at a particular reservoir for generating wind setup and run-up. The determination will generally include the results of meteorological studies that combine wind velocity, duration, direction and seasonal distribution characteristics in realistic manner.

Deterministic methodology. A method in which the chance of occurrence of the variable involved is ignored and the method or model used is considered to follow a definite law of certainty, and not probability.

Diaphragm wall (membrane). A sheet, thin zone, or facing made of an impervious material such as concrete, steel, wood, plastic, etc. Also see core wall.

Dike. See saddle dam.

Discharge. Refers generally to the outflow and is used as a measure of the rate at which a volume of water passes a given point. Therefore, the use of this term is not restricted as to course or location, and it can be used to describe the flow of water from a pipe or a drainage basin.

Diversion channel, canal, or tunnel. A waterway used to divert water from its natural course. The term is generally applied to a temporary arrangement, e.g., to bypass water around a dam site during construction. "Channel" is normally used instead of "canal" when the waterway is short.

Drain, blanket. A layer of pervious material placed to facilitate drainage of the foundation and/or embankment.

Drain, chimney. A vertical or inclined layer of pervious material in an embankment to facilitate and control drainage of the embankment fill.

Drain, toe. A system of pipe and/or pervious material along the downstream toe of a dam used to collect seepage from the foundation and embankment and convey it to a free outlet.

Drainage area. The area that drains to a particular point on a river or stream.

Drainage curtain. A line of vertical wells or boreholes to facilitate drainage of the foundation and abutments and to reduce water pressure.

Drainage wells or relief wells. Vertical wells downstream of or in the downstream shell of an embankment dam to collect and control seepage through and under the dam. A line of such wells forms a drainage curtain.

Drawdown. The difference between a water level and a lower water level in a reservoir within a particular time. Used as a verb, it is the lowering of the water surface.

E.

Earthquake. A sudden motion or trembling in the earth caused by the abrupt release of accumulated stress along a fault.

Effective crest of the dam. The elevation of the lowest point on the crest (top) of the dam, excluding spillways.

Embankment zone. An area or part of an embankment dam constructed using similar materials and similar construction and compaction methods throughout.

Emergency. A condition that develops unexpectedly, which endangers the structural integrity of a dam and/or downstream human life or property, and requires immediate action.

Emergency Action Plan (EAP). A plan of action to be taken to reduce the potential for property damage and loss of life in an area affected by a dam failure or large flood.

Emergency gate. A standby or reserve gate used only when the normal means of water control is not available for use.

Emergency Management Agency. The state and local agencies responsible for emergency operations, planning, mitigation, preparedness, response, and recovery for all hazards. Names of emergency management agencies may vary such as: Division of Emergency Management, Comprehensive Emergency Management, Disaster Emergency Services, Civil Defense Agency, Emergency and Disaster Services.

Emergency repairs. Any repairs that are temporary in nature and that are necessary to preserve the integrity of the dam and prevent a failure of the dam.

End sill. An upright obstruction usually located at the downstream end of a stilling basin. The end sill can be solid or dentate and is used to reduce the length of the stilling basin by creating additional tailwater depth and to provide for scour control.

Energy dissipater. A device constructed in a waterway to reduce the kinetic energy of fast flowing water.

Entrance Channel. A channel which conveys water from the reservoir to the control structure of a spillway. Also Approach channel.

Erosion. The wearing away of a surface (bank, streambed, embankment, or other surface) by floods, waves, wind, or any other natural process.

Eutrophication. A process where more organic matter is produced than existing biological oxidization processes can consume. The increase in the nutrient levels of a lake or other body of water; this usually causes an increase in the growth of aquatic animal and plant life.

Extreme event. A term used commonly in the field of risk management for collectively describing emergencies and disasters. These are events with low probability and high consequence.

F.

Failure. See Dam, Failure.

Failure mode. A failure mode is the sequence of events that may cause failure or disrupt the function of the dam-reservoir system or part of it. This series of events is associated with a determined loading scenario and has a logical sequence, which starts with a main initial triggering event, is followed by a chain of development or propagation events and culminates in dam failure. A potential failure mode is a physically plausible process for dam failure resulting from an existing inadequacy or defect related to a natural foundation condition, the dam or appurtenant structures design, the construction, the materials incorporated, the operations and maintenance, or aging process, which may lead to an uncontrolled release of the reservoir.

Failure mode analysis (FMA). A procedure to determine which malfunction symptoms appear immediately before or after a failure of a critical parameter in a system or structure.

Failure Modes and Effects Analysis (FMEA). A systematic, proactive method for evaluating a process to identify where and how it might fail and to assess the relative impact of different failures, in order to identify the parts of the process that are most in need of change.

Failure mode effects and criticality analysis (FMECA). A procedure performed after a failure mode effects analysis to classify each potential failure effect according to its severity and probability of occurrence.

Failure potential assessment. A judgment of the potential for failure of an essential element within the expected life of the project. Five terms are used to describe the assessment: negligible, low, moderate, high, and urgent. A rating of negligible reflects the judgment that failure of the essential element is regarded as very unlikely; low reflects the judgment that failure is unlikely; moderate reflects the judgment that failure is possible and further data collection and/or analyses may be required; high reflects the judgment that failure is very probable; urgent reflects the judgment that failure is imminent.

Fatigue. The weakening of a material caused by repeated and alternating loads, may be used in conjunction with either static or dynamic loading.

Federal Emergency Management Agency (FEMA). Federal agency (in United States of America) responsible for enforcing the legislation for disaster and emergency planning and response.

Fetch. The-straight-line distance across a body of water subject to wind forces. The fetch is one of the factors used in calculating wave heights in a reservoir.

Filter (filter zone). One or more layers of granular material graded (either naturally or by selection) so as to allow seepage through or within the layers while preventing the migration of material from adjacent zones. The types of filters are: chimney, collar and diaphragm.

Flashboards. Structural members of timber, concrete, or steel placed in channels or on the crest of a spillway to raise the reservoir water level but intended to be quickly removed, tripped, or fail in the event of a flood.

Flip bucket. An energy dissipater located at the downstream end of a spillway and shaped so that water flowing at a high velocity is deflected upwards in a trajectory away from the foundation of the spillway.

Flood. A temporary rise in water surface elevation resulting in inundation of areas not normally covered by water. Hypothetical floods may be expressed in terms of average probability of exceedance per year such as one-percent-chance-flood, or expressed as a fraction of the probable maximum flood or other reference flood.

Flood hydrograph. A graph showing, for a given point on a stream, the discharge, height, or another characteristic of a flood with respect to time.

Flood, Safety Evaluation (SEP). The largest flood for which the safety of a dam and appurtenant structure is to be evaluated.

Flood, Inflow Design (IDF). The flood flow above which the incremental increase in downstream water surface elevation due to failure of a dam or other water impounding structure is no longer considered to present an unacceptable threat to downstream life or property. The flood hydrograph used in the design of a dam and its appurtenant works particularly for sizing the spillway and outlet works and for determining maximum storage, height of dam, and freeboard requirements.

Flood, Probable Maximum (PMF). The flood that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the drainage basin under study.

Floods, according to IS-11223, Reaffirmed 2004, three definitions are used:

- ***Flood, Standard Project (SPF)*:** It is the flood that may be expected from the most severe combination of hydrological and meteorological factors that are considered reasonably characteristic of the region and is computed by using the Standard Project Storm (SPS).

- ***Flood, Inflow Design (IDF), for the Safety of the Dam.*** It is the flood for which, when used with standard specifications of other factors, the performance of the dam should be safe against overtopping, structural failure and the spillway and its energy dissipation arrangements, if provided for a lower flood, should function reasonably well.

- ***Flood, Inflow Design (IDF), for efficient operation of energy dissipation works:*** It 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. t is the flood for which, when used with standard.

Flood plain. An area adjoining a body of water or natural stream that may be covered by floodwater. Also, the downstream area that would be inundated or otherwise affected by the failure of a dam or by large flood flows. The area of the flood plain is generally delineated by a frequency (or size) of flood.

Flood routing. A process of determining progressively over time the amplitude of a flood wave as it moves past a dam or downstream to successive points along a river or stream.

Flood storage. The retention of water or delay of runoff either by planned operation, as in a reservoir, or by temporary filling of overflow areas, as in the progression of a flood wave through a natural stream channel.

Flow deflector. A device placed across the downstream portion of a stilling basin to change the flow pattern within the basin. A flow deflector is used to reduce or eliminate abrasion erosion damage caused by waterborne materials and debris within the stilling basin.

Fluctuation. The variation in water level, up or down, as a result of project operation.

Flume. An open channel constructed with masonry, concrete or steel of rectangular or U shaped cross section and designed for medium or high velocity flow. Also, a channel in which water is accelerated for purposes of measurement.

Foundation. The portion of the valley floor that underlies and supports the dam structure.

Fracture (joint). Crack or break in rocks along which no movement has occurred.

Freeboard. Vertical distance between a specified still water (or other) reservoir surface elevation and the top of the dam, without camber. For example, freeboard above maximum surface or free-board above normal reservoir level.

Flush bars. Or Flush escape Spillway. The spill channel starts with its bed level at FRL of the reservoirs. To protect erosion of spill channel bed near the reservoir rim, a concrete masonry sill is constructed flush with the spill channel bed. Hence the name.

Full Reservoir Level (FRL). The highest reservoir level which can be maintained without spillway discharge or without passing water through under sluices. This level is also called highest controlled water level. (IS 7323:1994 and IS-4410 Reaffirmed 2004).

G.

Gabion. A wire basket, filled with stones, used to stabilize banks of a water course.

Gallery. A passageway in the body of a dam used for inspection, foundation grouting, and/or drainage.

Gantry crane. A fixed or traveling bent-supported crane for handling heavy equipment.

Gate. A movable water barrier for the control of water.

Bascule gate. See flap gate.

Bulkhead gate. A gate used either for temporary closure of a channel or conduit before dewatering it for inspection or maintenance or for closure against flowing water when the head difference is small, e.g., for diversion tunnel closure.

Crest gate (spillway gate). A gate on the crest of a spillway to control the discharge or reservoir water level.

Drum gate. A type of spillway gate consisting of a long hollow drum. The drum may be held in its raised position by the water pressure in a flotation chamber beneath the drum.

Emergency gate. A standby or auxiliary gate used when the normal means of water control is not available. Sometimes referred to as guard gate.

Fixed wheel gate (fixed roller gate) (fixed axle gate). A gate having wheels or rollers mounted on the end posts of the gate. The wheels bear against rails fixed in side grooves or gate guides.

Flap gate. A gate hinged along one edge, usually either the top or bottom edge. Examples of bottom-hinged flap gates are tilting gates and fish belly gates so called from their shape in cross section.

Flood gate. A gate to control flood release from a reservoir.

Outlet gate. A gate controlling the flow of water through a reservoir outlet.

Radial gate (Tainter gate). A gate with a curved upstream plate and radial arms hinged to piers or other supporting structure.

Regulating gate (regulating valve). A gate or valve that operates under full pressure flow conditions to regulate the rate of discharge.

Roller drum gate. See drum gate.

Roller gate (stoney gate). A gate for large openings that bears on a train of rollers in each gate guide.

Skimmer gate. A gate at the spillway crest whose prime purpose is to control the release of debris and logs with a limited amount of water. It is usually a bottom hinged flap or Bascule gate.

Slide gate (sluice gate). A gate that can be opened or closed by sliding in supporting guides.

Gate chamber (valve chamber). A room from which a gate or valve can be operated, or sometimes in which the gate is located.

Geomembrane. An impermeable geosynthetic composed of one or more synthetic sheets.

Geosynthetic. A planar product manufactured from a polymeric material used with soil, rock, earth, or other geotechnical engineering related material as an integral part of a project, structure, or system.

Geotextil. Any fabric or textile (natural or synthetic) when used as an engineering material in conjunction with soil, foundations, or rock. Geotextiles have the following uses: drainage, filtration, separation of materials, reinforcement, moisture barriers, and erosion protection.

Glacis. A straight or curved floor sloping below and in continuation of the raised crest or sill of a weir or fall structure.

GLOF. Glacial Lake Outburst Flood. The flood that occurs when the water dammed by a glacier or a moraine is released suddenly.

Gradation: The distribution of particles of granular material among standard sizes, usually expressed in terms of cumulative percentages larger or smaller than each of a series of sieve openings.

Groin. The area along the contact (or intersection) of the face of a dam with the abutments.

Grout. A fluidized material that is injected into soil, rock, concrete, or other construction material to seal openings and to lower the permeability and/or provide additional structural strength. There are four major types of grouting materials: chemical; cement; clay; and bitumen.

Grout blanket. An area of the foundation systematically grouted to a uniform shallow depth.

Grout cap. A concrete filled trench or pad encompassing all grout lines constructed to impede surface leakage and to provide anchorage for grout connections.

Grout curtain. One or more zones, usually thin, in the foundation into which grout is injected to reduce seepage under or around a dam.

H.

Hazard. A situation that creates the potential for adverse consequences such as loss of life, property damage, or other adverse impacts.

Hazard potential. The possible adverse incremental consequences that result from the release of water or stored contents due to failure of the dam or misoperation of the dam or appurtenances. Impacts may be for a defined area downstream of a dam from flood waters released through spillways and outlet works of the dam or waters released by partial or complete failure of the dam. There may also be impacts for an area upstream of the dam from effects of backwater flooding or landslides around the reservoir perimeter.

Hazard potential classification. A system that categorizes dams according to the degree of adverse incremental consequences of a failure or misoperation of a dam. The hazard potential classification does not reflect in any way on the current condition of the dam (i.e., safety, structural integrity, flood routing capacity).

Head, static. The vertical distance between two points in a fluid.

Headwater: The water immediately upstream from a dam. The water surface elevation varies due to fluctuations in inflow and the amount of water passed through the dam.

Heel. The junction of the upstream face of a gravity or arch dam with the ground surface. For an embankment dam, the junction is referred to as the upstream toe of the dam.

Hydraulic head. Piezometric head. It is a specific measurement of liquid pressure above a vertical datum. It is usually measured as a liquid surface elevation, expressed in units of length.

Head, total or energy head. In a fluid is the sum of kinetic and potential energies. Recall that potential energies are pressure energy and elevation energy.

Head, velocity. Kinetic energy. The vertical distance that would statically result from the velocity of a moving fluid.

Headwater: The water immediately upstream from a dam. The water surface elevation varies due to fluctuations in inflow and the amount of water passed through the dam.

Heel. The junction of the upstream face of a gravity or arch dam with the ground surface. For an embankment dam, the junction is referred to as the upstream toe of the dam.

Height, above ground. The maximum height from natural ground surface to the top of a dam.

Height, hydraulic. The vertical difference between the maximum design water level and the lowest point in the original streambed.

Height, structural. The vertical distance between the lowest point of the excavated foundation to the top of the dam.

Hoist. The mechanism by which a bucket or blade is lifted, or the process of lifting it.

Hydraulic fracturing. Hydraulic fracturing in soils is a tensile parting that is created because of increased fluid pressure. Initiation and/or propagation cracks in the core sections of earthen dams because of hydraulic fracturing affect adversely structural safety of the dams.

Hydraulic gradient. The change in total hydraulic pressure per unit distance of flow. Hydrology – One of the earth sciences that encompasses the natural occurrence, distribution, movement, and properties of the waters of the earth and their environmental relationships.

Hydraulic jump. The abrupt rise in a water surface when flow at high velocity (supercritical) discharges into a zone of lower velocity (subcritical). The hydraulic jump is a commonly used method of energy dissipation.

Hydrograph, breach or dam failure. A flood hydrograph resulting from a dam breach.

Hydrograph, flood. A graph showing, for a given point on a stream, the discharge, height, or other characteristic of a flood with respect to time.

Hydrograph, unit. A hydrograph with a volume of one inch of runoff resulting from a storm of a specified duration and areal distribution. Hydrographs from other storms of the same duration and distribution are assumed to have the same time base but with ordinates of flow in proportion to the runoff volumes.

Hydrology. One of the earth sciences that encompasses the natural occurrence, distribution, movement, and properties of the waters of the earth and their environmental relationships.

Hydrometeorology. The study of the atmospheric and land-surface phases of the hydrologic cycle with emphasis on the interrelationships involved.

Hydro-mechanical Equipment. Gates, valves, hoists, and elevators.

I.

Impact basin. An energy dissipating structure that utilizes a positive barrier (baffle) within the flow area. Energy dissipation is accomplished by the turbulence created by the loss of momentum as the flow impacts the barrier and the direction of flow is changed.

Inflow Design Flood (IDF). See Flood.

Instrumentation. An arrangement of devices installed into or near dams that provide for measurements that can be used to evaluate the structural behavior and performance parameters of the structure.

Intake. Placed at the beginning of an outlet-works waterway (power conduit, water supply conduit), the intake establishes the ultimate drawdown level of the reservoir by the position and size of its opening(s) to the outlet works. The intake may be vertical or inclined towers, drop inlets, or submerged, box-shaped structures. Intake elevations are determined by the head needed for discharge capacity, storage reservation to allow for siltation, the required amount and rate of withdrawal, and the desired extreme draw-down level.

Internal erosion (also piping). A general term used to describe all the various erosional processes where water moves internally through or adjacent to the soil zones of embankment dams and foundation, except for the specific process referred to as backward erosion piping.

The term internal erosion is used in place of a variety of terms that have been used to describe various erosional processes, such as scour, suffusion, concentrated leak piping, and others.

Inundation map. A map showing areas that would be affected by flooding from releases from a dam's reservoir. The flooding may be from either controlled or uncontrolled releases or as a result of a dam failure. A series of maps for a dam could show the incremental areas flooded by larger flood releases.

L.

Landslide. The unplanned descent (movement) of a mass of earth or rock down a slope.

Large dam: two definitions.

Large dam (actual definition according to ICOLD´s Constitution Status, July 2011). A dam with a height of 15 meters or greater from lowest foundation to crest or a dam between 5 meters and 15 meters impounding more than 3 million cubic meters

Large dam (definition used by several dam organizations and IS-4410 Part 8). A dam that is above 15 meters in height, measured from the lowest part of the general foundation area to the top of dam; or a dam between 10 to 15 meters in height and that satisfies at least one of the following, namely

a) The length of crest is not less than 500 meters;

b) The capacity of the reservoir formed by the dam is not less than one million cubic meters;

c) The maximum flood discharge dealt with by the dam is not less than 2000 cubic meters per second;

d) The dam has particularly difficult foundation problems; or

e) The dam is of unusual design.

Leakage. Uncontrolled loss of water by flow through a hole or crack.

Length of dam. The length along the top of the dam. This also includes the spillway, powerplant, navigation lock, fish pass, etc., where these form part of the length of the dam. If detached from the dam, these structures should not be included.

Level of protection (LOP). As used in the evaluation process, it is the level of loading selected to which corrective actions will be designed to prevent dam failure.

Lining. With reference to a canal, tunnel, shaft, or reservoir, a coating of asphaltic concrete, reinforced or unreinforced concrete, shotcrete, rubber or plastic to provide watertightness, prevent erosion, reduce friction, or support the periphery of the outlet pipe conduit.

Loading scenario. To obtain the risk associated with a dam, the calculation is usually disaggregated into various scenarios, depending on the event that originates failure. For instance, a dam may fail when subjected to a flooding or to an earthquake, and it is convenient to do those calculations in a separate way, each situation being called loading scenario.

Logboom. A chain of logs, drums, or pontoons secured end-to-end and floating on the surface of a reservoir so as to divert floating debris, trash, and logs.

Loss of life. Human fatalities that would result from a failure of the dam, without considering the mitigation of loss of life that could occur with evacuation or other emergency actions.

Low level outlet (bottom outlet). An opening at a low level from a reservoir generally used for emptying or for scouring sediment and sometimes for irrigation releases.

M.

Maintenance. Those tasks that are generally recurring and are necessary to keep the dam and appurtenant structures in a sound condition and free from defect or damage that could hinder the dam's functions as designed, including adjacent areas that also could affect the function and operation of the dam.

Maintenance inspection. Visual inspection of the dam and appurtenant structures by the owner or owner's representative to detect apparent signs of deterioration, other deficiencies, or any other areas of concern.

Maximum flood control level. The highest elevation of the flood control storage.

Maximum Operating Pool Elevation or Maximum Operation Level. The upper limit or top of active storage. This is the reservoir elevation that would be attained when the reservoir is fully utilized for all purposes, including flood control. It represents the highest elevation achieved in the reservoir under normal operating conditions.

Maximum Storage Capacity. The volume, in cubic hectometers (Hm^3), of the impoundment created by the dam at the effective crest of the dam; only water that can be stored above natural ground level or that could be released by failure of the dam is considered in assessing the storage volume; the maximum storage capacity may decrease over time due to sedimentation or increase if the reservoir is dredged.

Maximum Water Level (MWL): The maximum water level likely to be attained in reservoir at the dam face while negotiating the adopted design flood. It also depends on specified initial reservoir level and the spillway regulation rule. Also "Highest Reservoir Level". (IS-4410 Reaffirmed 2004).

Maximum wind. The most severe wind for generating waves that is reasonably possible at a particular reservoir. The determination will generally include results of meteorologic studies that combine wind velocity, duration, direction, fetch, and seasonal distribution characteristics in a realistic manner.

Meteorological homogeneity. Climates and orographic influences that are alike or similar.

Meteorology. The science that deals with the atmosphere and atmospheric phenomena, the study of weather, particularly storms and the rainfall they produce.

Minimum operating level. The lowest level to which the reservoir is drawn down under normal operating conditions. The lower limit of active storage.

Multipurpose project. A project designed for irrigation, power, flood control, municipal and industrial, recreation, and fish and wildlife benefits, in any combinations of two or more. Contrasted to single-purpose projects serving only one need.

N.

Nappe. A sheet of water overflowing a weir, fall, etc. The nappe has upper and lower surfaces.

Non-overflow dam (section). A dam or section of dam that is not designed to be overtopped.

Normal reservoir level. For a reservoir with a fixed overflow sill the lowest crest level of that sill. For a reservoir whose outflow is controlled wholly or partly by moveable gates, siphons or other means, it is the maximum level to which water may rise under normal operating conditions, exclusive of any provision for flood surcharge.

Normal storage capacity. The volume, in millions of cubic meters (Mm3), of the impoundment created by the dam at the lowest uncontrolled spillway crest elevation, or at the maximum elevation of the reservoir at the normal (non-flooding) operating level.

Notification. To inform appropriate individuals about an emergency condition so they can take appropriate action.

O.

Ogee. See Weir Ogee.

One-Percent-Chance Flood. A flood that has 1 chance in 100 of being equaled or exceeded during any year.

Orographic. Physical geography that pertains to mountains and to features directly connected with mountains and their general effect on storm path and generation of rainfall.

Outlet. An opening for releasing discharge that is lower than the spillway crest. Designed to release reservoir water through or around a dam. An opening through which water can be freely discharged from a reservoir to the river for a purpose. A conduit or pipe controlled by a gate or valve, or a siphon, that is used to release impounded water from the reservoir.

Outlet Channel. A channel connecting the stilling basin at the downstream end of the spillway to the river channel or other drainage channel below the dam.

Outlet works. A dam appurtenance that provides release of water (generally controlled) from a reservoir. These works are a combination of structures and equipment required for the safe operation and control of water released from a reservoir to serve various purposes, i.e., regulate stream flow and quality; release floodwater; and provide irrigation, municipal, and/or industrial water. Included in the outlet works are the intake structure, conduit, control house-gates, regulating gate or valve, gate chamber, and stilling basin. 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.

Overflow dam (section). A section or portion of a dam designed to be overtopped.

Overtopping failure. A hydrologic dam failure that occurs because of the water level in the reservoir exceeding the height of the dam.

P.

Parapet wall. A solid wall built along the top of a dam (upstream or downstream edge) used for ornamentation, for safety of vehicles and pedestrians, or to prevent overtopping caused by wave runup.

Peak flow. The maximum instantaneous discharge that occurs during a flood. It is coincident with the peak of a flood hydrograph.

Penstock. A pressurized pipeline or shaft between the reservoir and hydraulic machinery.

Pervious zone. A part of the cross section of an embankment dam comprising material of high permeability.

Phreatic surface. The free surface of water seeping at atmospheric pressure through soil or rock.

Piezometer. An instrument used for measure water levels or pore water pressures in embankments, foundations, abutments, soil, rock, or concrete. (See instrumentation.)

Piping. The progressive development of internal erosion by seepage.

Piping failure. Dam failure caused when concentrated seepage develops within an embankment dam and erodes to form a "pipe." Piping typically occurs in two phases: formation of the "pipe" and the subsequent collapse of the dam crest. It is possible for the reservoir to drain before the dam crest collapses.

Pitting: A form of localized corrosive attack characterized by holes in metal. Depending on the environment and the material, a pit may take months, or even years, to become visible.

Plunge basin: A deep pool into which a free jet of water discharges for the purpose of kinetic energy dissipation before being returned to the downstream channel.

Plunge pool. A natural or artificially created deep pool that dissipates the kinetic energy of free water jet, before being returned to the downstream channel.

Population at Risk (PAR): the estimated number of people whose safety will be at risk if the dam, or the proposed dam after its construction, fails. This figure refers to people at any place within inundation zones that will get their feet wet and corresponds to those who are not at risk by a flood event (natural or pre dam condition) but are at risk when the same flood event is accompanied by a dam failure event. Also (in some references)as "dam break PAR".

Pressure relief pipes. Pipes used to relieve uplift or pore water pressure in a dam foundation or in the dam structure.

Probability. The likelihood of an event occurring.

Probable. Likely to occur; reasonably expected; realistic.

Probable Maximum Flood (PMF). See Flood.

Probable Maximum Precipitation (PMP). Theoretically, the greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographical location during a certain time of the year.

R.

Rehabilitation. Two definitions:

a) ***Rehabilitation (as used by dam and emergency organizations).*** Work that aims to restore the service life of a structure, as opposed to maintenance, which seeks to restore the status quo, and upgrading whose purpose is to maximize the performance within the physical limits of the structure. It is not intended to enhance performance beyond its original level.

b) ***Rehabilitation (as per CWC).*** Repair, replacement, reconstruction or removal of a dam that is carried out to meet applicable dam safety standards. It is similar to remedial measures.

Reliability. For gate and mechanical systems reliability is defined as the likelihood of successful performance of a given project element. It may be measured on an annual basis or for some specified period of interest or, for example, in the case of spillway gates, on a per demand basis. Mathematically, Reliability = 1 - Probability of unsatisfactory operation.

Remote operation. Operation of mechanical features from an on-site location other than at the feature.

Remotely operated vehicle (ROV). An unoccupied, highly maneuverable underwater robot controlled by a remote operator usually located in a ship or on the shore. Most vehicles are equipped with a video camera and lights. Additional equipment can be added to expand the vehicle's capabilities.

Repairs. Any work done on a dam that may affect the integrity, safety, and operation of the dam.

Reservoir. A body of water impounded by a dam and in which water can be stored.

Reservoir Area. The total surface of a reservoir measured in a horizontal plane at an elevation corresponding to the full supply level of the reservoir. The area that would be flooded due to backwater elevations or surcharge is not included.

Reservoir Capacity. The total volume of water a reservoir can hold when filled up to the full sup-ply or normal water level. Storage derived from temporary flashboards, surcharge, or backwater curve is not included. Reservoir capacity usually is reported as of the date of construction of the dam. The sum of the dead and live storage of the reservoir.

Reservoir Storage. The retention of water or delay of runoff in a reservoir either by the planned operation, as in a reservoir, or by temporary filling in the progression of a flood wave. See Storage.

Reservoir regulation procedure (Rule Curve): The compilation of operating criteria, guidelines, and specifications that govern the storage and release function of a reservoir. It may also be referred to as operating rules, flood control diagram, or water control schedule. These are usually expressed in the form of graphs and tabulations, supplemented by concise specifications and are often incorporated in computer programs. In general, they indicate limiting rates of reservoir releases required or allowed during various seasons of the year to meet all functional objectives of the project.

Reservoir rim. The boundary of the reservoir including all areas along the valley sides above and below the water surface elevation associated with the routing of the IDF.

Reservoir surface area. The area covered by a reservoir when filled to a specified level.

Return period. The average time interval between occurrences of a hydrological event of a given magnitude or greater, usually expressed in years.

Revetment. An embankment or wall of sandbags, earth, etc., constructed to restrain material from being transported away. A facing of stone, cement, sandbags, etc., to protect a wall or embankment.

Riprap. A layer of large uncoursed stone, precast blocks, bags of cement, or other suitable material, generally placed on the slope of an embankment or along a watercourse as protection against wave action, erosion, or scour. Riprap is usually placed by dumping or other mechanical methods, and in some cases is hand placed. It consists of pieces of relatively large size, as distinguished from a gravel blanket.

Risk. A measure of the likelihood and severity of adverse consequences (National Research Council 1983). Risk is the combination of three concepts: what can happen, how likely it is to happen and what are the consequences in the case it happens.

Risk analysis. A procedure to identify and quantify risks by establishing potential failure modes, providing numerical estimates of the likelihood of an event in a specified time period, and estimating the magnitude of the consequences.

Risk assessment. The process of deciding whether existing risks are tolerable and present risk control measures are adequate and, if not, whether alternative risk control measures are justified. Risk assessment incorporates the risk analysis and risk evaluation phases.

Risk management. A structured approach to understanding the nature of the hazards posed by the design, construction or operation of project works. The organization of the decisions made in the light of the perceived hazards.

River basin. The drainage area for a river above a particular point.

Robustness: The quality or condition of being strong and in good condition. The ability to withstand or overcome adverse conditions or rigorous testing.

Rock anchor. A steel rod or cable placed in a hole drilled in rock, held in position by grout, mechanical means, or both. In principle, the same as a rock bolt, but usually the rock anchor is more than 4 meters long.

Rock reinforcement. The placement of rock bolts, untensioned rock dowels, prestressed rock anchors, or wire tendons in a rock mass to reinforce and mobilize the rock's natural competency to support itself.

Runup. The vertical distance above the setup that the rush of water reaches when a wave breaks on the dam embankment.

S.

Saddle dam (or dike). A subsidiary dam of any type constructed across a saddle or low point on the perimeter of a reservoir.

***Scour*.** The loss of material occurring at an erosional surface, where a concentrated flow is found, such as a crack in a dam or the dam/foundation contact. Continued flow causes the erosion to progress, creating a larger and larger eroded area. Commonly applied to the streambed surface.

Sediment. Any finely divided organic and/or mineral matter deposited by air or water in nonturbulent areas. Unconsolidated solid material that comes from weathering of rock and is carried by, suspended in, or deposited by water or wind.

Sedimentation. Deposition of waterborne sediments due to a decrease in velocity and corresponding reduction in the size and amount of sediment which can be carried.

Seepage. The internal movement of water that may take place through the dam, the foundation or the abutments.

Seiche. An oscillating wave in a reservoir caused by a landslide into the reservoir or earthquake-induced ground accelerations or fault offset or meteorological event.

Sensitivity analysis. An analysis in which the relative importance of one or more of the variables thought to have an influence on the phenomenon under consideration is determined.

Settlement. The vertical downward movement of a structure or its foundation.

***Shotcrete*.** Concrete sprayed through a nozzle onto the surface to be covered.

Shoulder (shell). The upstream and downstream parts of the cross section of an embankment dam on each side of the core or core wall. Hence the expression upstream shoulder or downstream shoulder.

Sinkhole. A depression that indicates subsurface settlement or particle movement, typically having clearly defined boundaries with a sharp offset.

Significant wave height. Average height of the one-third highest individual waves. May be estimated from wind speed, fetch length, and wind duration.

Siphon. An inverted U-shaped pipe or conduit, filled until atmospheric pressure is enough to force water from a reservoir over an embankment dam and out of the other end.

Sill. Horizontal overflow line of a measuring notch or spillway. Also a horizontal member on which a gate rests when closed.

Slide. Movement of a mass of earth down a slope on the embankment or abutment of a dam.

Slope. Inclination from the horizontal. Sometimes referred to as batter when measured from vertical.

Slope protection. The protection of a slope against wave action or erosion. See Riprap.

Slip lining: The process of inserting a new, smaller-diameter lining or pipe into an existing larger-diameter conduit.

Sluice. An opening for releasing water from below the static head elevation. A sluice is a water conduit controlled at its head by a gate.

Slurry trench. A trench cut into an embankment or its foundation and filled with a flexible watertight slurry to prevent the passage of water.

Spillway. A structure over or through which flow is discharged from a reservoir. If the rate of flow is controlled by mechanical means, such as gates, it is considered a controlled spillway. If the geometry of the spillway is the only control, it is considered an uncontrolled spillway.

Spillway, auxiliary. According to FEMA. Any secondary spillway that is designed to be operated infrequently, possibly in anticipation of some degree of structural damage or erosion to the spillway that would occur during operation.

Spillway auxiliary. According to IS-4410 Part 9. See Emergency Spillway.

Auxiliary spillway. According to USBR. A spillway, usually located in a saddle or depression in the reservoir rim which leads to a natural or excavated waterway, located away from the dam which permits the planned release of excess flood flow beyond the capacity of the service spillway. A control structure is seldom furnished. The crest is set at the maximum water surface elevation for a 100-year flood or some other specific frequency flood. The auxiliary spillway thus has only infrequent use.

Spillway, emergency. According to IS-4410 Part 9: An excavated or natural channel, usually some distance away from a dam, provided to permit the release of extra ordinary 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 of the reservoir to rise above the crest of the emergency spillway. If continued in flow causes overtopping of the embankment plug, it is intended that the plug shall wash away, releasing the excess water without endangering the main dam.

Emergency spillway. According to USBR. A spillway which provides for additional safety should emergencies not contemplated by normal design assumptions be encountered, i.e., inoperable outlet works, spillway gates, or spillway structure problems. The crest is usually set at maximum water surface. A spillway that is designed to provide additional protection against overtopping of a dam and is intended for use under extreme conditions such as misoperation or malfunction of the service spillway or other emergency conditions.

Emergency Spillway. According to IS-4410 Part 9. An excavated or natural channel, usually some distance away from a dam, provided to permit the release of extra ordinary 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 of the reservoir to rise above the crest of the emergency spillway. If continued in flow causes overtopping of the embankment plug, it is intended that the plug shall wash away, releasing the excess water without endangering the main dam. It is also called "Auxiliary Spillway".

Spillway, service. According to FEMA. A spillway that is designed to provide continuous or frequent regulated or unregulated releases from a reservoir, without significant damage to either the dam or its appurtenant structures. This is also referred to as principal spillway.

Service Spillway. According to IS-4410 Part 9. Any spillway that is normally utilized to discharge surplus waters. The downstream channel is protected by paving, so that it is not damaged due to high impact and velocities of the water. Also "Main Spillway".

Spillway capacity: The maximum spillway outflow that a dam can safely pass with the reservoir at its maximum level. According to IS-11223: Capability of the spillway, as determined by its dimensions, crest level and hydraulic characteristics in disposing off water at any specific level.

***Spillway, cascade**. A* spillway having a series of steps on the downstream face of the spillway for dissipation of energy by contact with successive steps. It is also called "Stepped Spillway".

Spillway channel. An open channel or closed conduit conveying water from the spillway inlet downstream.

Spillway, chute. A steeply sloping spillway channel that conveys discharges at supercritical velocities.

Spillway Components. Parts of a spillway which convey, control, carry away, discharge, destroy excess energy and finally lead water to an outlet channel and comprising all or some of the following: (a) entrance channel, (b) control structure, (c) discharge carrier, (d) energy dissipator, and (e) spillway outlet channel.

Spillway, Controlled. A spillway having means to control the outflow from the reservoir. It is also called "Gated Spillway".

Spillway, Uncontrolled. A spillway, the crest of which permits water to escape automatically, as the water level in the reservoir rises above the crest. It is also called 'Ungated Spillway'.

Spillway, crest. The lowest level at which water can flow over or through the spillway.

Spillway Design Flood (SDP). See Flood, Inflow Design.

Spillway, fuse plug. A form of auxiliary spillway consisting of a low embankment designed to be overtopped and washed away during an exceptionally large flood.

Spillway, Overfall (Straight Drop Spillway). A spillway constructed on small bunds designed to permit water to pass over its crest. It is a low weir and is a simple vertical fall type structure.

Spillway, Overflow (Ogee Spillway). It is an improvement upon the free overfall spillway for higher depths of water and is widely used with concrete, masonry, arch and buttress dams. The profile of the spillway is made in accordance with the lower nappe of a free falling jet over a duly ventilated sharp crested weir for the design head.

Spillway Side Channel. A spillway in which the initial and final flow are approximately at right angle to each other. The water, after passing over the spillway weir or ogee crest, being carried away by a channel running essentially parallel to the spillway crest

Spillway, shaft. A vertical or inclined shaft into which water spills and then is conveyed through, under, or around a dam by means of a conduit or tunnel. If the upper part of the shaft is splayed out and terminates in a circular horizontal weir, it is termed a bell-mouth or morning glory spillway. Also "Drop inlet" and "Morning Glory spillway".

Spillway, stepped. See spillway cascade.

Spillway, tunnel. A tunnel used as an outlet channel for a side channel spillway.

Splitter wall. A wall parallel to the direction of flow in a chute or stilling basin that separates flows released from different sources as a means of energy dissipation.

Spray lining: The application of cement mortar or epoxy resin against the inside walls of an existing conduit, using a revolving spray head moved through the conduit

Stability. The condition of a structure or a mass of material when it is able to support the applied stress for a long time without suffering any significant deformation or movement that is not reversed by the release of the stress.

Stagnation pressure: A static pressure at a stagnation point in a fluid flow. At a stagnation point the fluid velocity is zero and all kinetic energy has been converted into pressure energy.

Stilling basin. A basin constructed to dissipate the energy of rapidly flowing water, e.g., from a spillway or outlet, and to protect the riverbed from erosion.

Stillwater level. The elevation that a water surface would assume if all wave actions were absent.

Stilling pool. A pool located below a spillway, gate, or valve into which the discharge dissipates energy to avoid downstream channel degradation. An unlined stilling basin usually constructed in natural ground or rock.

Stoplogs. Large logs, timbers, or steel beams placed on top of each other with their ends held in guides on each side of a channel or conduit so as to provide a cheaper or more easily handled means of temporary closure than a bulkhead gate.

Storage. The retention of water or delay of runoff either by planned operation, as in a reservoir, or by temporary filling of overflow areas, as in the progression of a flood wave through a natural stream channel. Definitions of specific types of storage in reservoirs are:

Active storage. The volume of the reservoir that is available for some use such as power generation, irrigation, flood control, water supply, etc. The bottom elevation is the minimum operating level.

Dead storage. The storage that lies below the invert of the lowest outlet and that, therefore, cannot readily be withdrawn from the reservoir.

Flood surcharge. The storage volume between the top of the active storage and the design water level.

Inactive storage. The storage volume of a reservoir between the crest of the invert of the lowest outlet and the minimum operating level.

Live storage. The sum of the active-and the inactive storage.

Reservoir capacity. The sum of the dead and live storage of the reservoir.

Surcharge. The volume or space in a reservoir between the controlled retention water level and the maximum water level. Flood surcharge cannot be retained in the reservoir but will flow out of the reservoir until the controlled retention water level is reached.

Surge. A rapid increase in the depth of flow.

T.

Tailwater. The water immediately downstream from a dam. The water surface elevation varies due to fluctuations in the outflow from the structures of a dam and due to downstream influences of other dams or structures.

Terminal structure. Portion of spillway downstream from chute, tunnel or conduit, which generally dissipates or stills releases. Concrete portion of an outlet works downstream from a conduit, tunnel, or control structure. The structure dissipates or stills releases. See stilling basin.

Thrust block. A massive block of concrete built to withstand a thrust or pull.

Toe drain. A system of pipe and/or pervious material along the downstream toe of a dam used to collect seepage from the foundation and embankment and convey it to a free outlet.

Toe of the dam. The junction of the downstream slope or face of a dam with the ground surface; also referred to as the downstream toe. The junction of the upstream slope with ground surface is called the heel or the upstream toe.

Topographic map. A detailed graphic delineation (representation) of natural and man-made features of a region with particular emphasis on relative position and elevation.

Top thickness (top width). The thickness or width of a dam at the level of the top of dam (excluding corbels or parapets). In general, the term thickness is used for gravity and arch dams, and width is used for other dams.

Trashrack. A device located at an intake to prevent floating or submerged debris from entering the intake.

Training wall. A wall built to confine or guide the flow of water.

Tributary. A stream that flows into a larger stream or body of water

Tunnel. A long underground excavation with two or more openings to the surface, usually having a uniform cross section used for access, conveying flows, etc.

U.

Undermining. Dig or excavate beneath a building so as to make it collapse.

Upgrading. It is the work considered necessary to maximize the benefit of an existing installation. New construction or equipment is installed where it can be justified economically.

Uplift. The hydrostatic force of water exerted on or underneath a structure, tending to cause a displacement of the structure.

Upstream blanket. An impervious blanket placed on the reservoir floor and abutments upstream of a dam. For an embankment dam, the blanket may be connected to the core.

Unit Hydrograph. See Hydrograph, unit.

V.

Valve. A device fitted to a pipeline or orifice in which the closure member is either rotated or moved transversely or longitudinally in the waterway so as to control or stop the flow.

> ***Butterfly valve.*** A valve designed for quick closure that consists of a circular leaf, slightly convex in form, mounted on a transverse shaft carried by two bearings.
>
> ***Check valve.*** Any device which will allow fluid or air to pass through it in only one direction. A special valve with a hinged disc or flap that opens in the direction of normal flow and is forced shut when flows attempt to go in the reverse or opposite direction of normal flow. A device preventing backflow in pipes. Water can flow readily in one direction but any reversal of the flow causes the check valve to close.

Fixed-cone valve (Howell Bunger valve). A free discharge valve for regulating high-pressure outlets from dams and reservoirs. Inside the valve there is a fixed-cone, pointed upstream, which ensures dispersion of the jet. Outside the valve, a cylindrical sleeve moves downstream to shut off flow by sealing on the periphery of the cone. The discharged jet spreads out in a wide cone angle and breaks up into a fine spray. A containment jet is possible by using a hood.

Gate valve. A valve with a circular-shaped closing element that fits securely over an opening through which water flows. A valve that utilizes a disc moving at a right angle to the flow to regulate the rate of flow. When a gate valve is fully opened, there is no obstruction to the flow.

Hollow jet valve. A device for regulating high-pressure outlets. Essentially, it is half a needle valve in which the needle closure member moves upstream toward the inlet end of the valve to shut off flow. As there is no convergence at the outlet end, the flow emerges in the form of an annular cylinder, segmented by several splitter ribs for admitting air into the jet interior to prevent jet instability.

Needle valve. Any of a family of valves which regulate flow through the use of a needle moving into and out of an orifice. Types of this valve include: the Ensign, Balanced Needle, Internal Differential Needle, and Interior Differential Needle.

Relief valve. A valve which will allow air or fluid to escape if its pressure becomes higher than the valve setting. A safety device that automatically provides protection against excessive temperatures, excessive pressures, or both.

Sleeve valve. A valve designed to incorporate multiple tapered nozzles on the sleeve for controlling flow and reducing pressure. The sleeve valve is capable of dissipating energy enabling them to be opened against high differential head without damaging the seals.

Tube valve. A valve which is opened or closed by mechanically moving a tube upstream or downstream by an actuating screw.

Vortex. A revolving mass of water (whirlpool) in which the streamlines are concentric circles and in which the total head is the same. Water rotating about an axis.

W.

Watershed. The area drained by a river or river system or portion thereof. The watershed for a dam is the drainage area upstream of the dam.

Watershed divide. The divide or boundary between catchment areas (or drainage areas).

Water hammer (hydraulic transient). Refers to pressure fluctuations caused by a sudden increase or decrease in flow velocity, usually associated with a rapid closure or opening of a valve in a pipeline.

Waterways. Spillways and outlet works.

Wave protection. Riprap, concrete, or other armoring on the upstream face of an embankment dam to protect against scouring or erosion due to wave action.

Wave runup. Vertical height above the stillwater level to which water from a specific wave will run up the face of a structure or embankment.

Weir. A notch of regular form through which water flows.

Weir, broad-crested. An overflow structure on which the nappe is supported for an appreciable length in the direction of flow.

Weir, measuring. A device for measuring the rate of flow of water. It generally consists of a rectangular, trapezoidal, triangular, or other shaped notch, located in a vertical, thin plate over which water flows. The height of water above the weir crest is used to determine the rate of flow.

Weir, ogee. A reverse curve, shaped like an elongated letter "S." The downstream faces of overflow spillways are often made to this shape. Also Ogee.

Wind setup. The vertical rise in the stillwater level at the face of a structure or embankment caused by wind stresses on the surface of the water.

Wing wall. A wall that guides a water into a conveyance structure.

References

Appendix A

1. CWC. 2019. "Guidelines for Assessing and Managing Risks Associated with Dams". Doc. No. CDSO_GUD_DS_10_v1.0
2. ICOLD. 2005. "Dam Safety Management", General Reporter Q91, 20° Large dams Congress, Brasilia, Brazil.
3. iPresas Risk Analysis, 2019. "Identificación de Modos de Falla de la presa de la presa de Santa Rita".
4. José María Carrillo Sánchez, 2009. "Evaluación de la seguridad hidrológico - hidráulica de la presa de Compuerto (T.M. de Velilla del rio Carrión, provincia de Palencia) perteneciente a la cuenca hidrográfica del Duero." Universidad politécnica de Valencia.

Appendix B.1

1. Castillejo, N., Marcano, A., 1988. "Aeration at Guri Final Stage Spillway", Model-Prototype correlation of HydraulicStructures, Proceedings of the International Symposium, ASCE, Colorado Springs.
2. Castillejo, N., Patiño, A., 1981. "Primeras Experiencias de Aireación en el Aliviadero de Guri ", III Jornadas Nacionales de Ingenierìa Hidráulica, Barquisimeto, Septiembre.
3. CVG- EDELCA, 1987. Dpto. de Hidráulica, "Reevaluación del Proyecto Guri ".
4. De Fazio, F., Castillejo, N., 1982. "Aeration of the Guri Spillway to prevent Cavitation Damage ", 14 th Congreso de Grandes Presas. Rio de Janeiro (no public ado).
5. Falvey, H., 1990."Cavitation in Chute and Spillways ", Bureau of Reclamation.
6. Falvey, H. T., 1983. "Prevention of cavitation in chutes and Spillways ", en Proceedings of the Conference on Frontiers in Hydraulic Engineers, MIT, Cambridge, August.
7. Falvey, H., 1979. "Air –Water flows in Hydraulic Structures", US Bureau of Reclamation.
8. Galperín, y Colaboradores, 1979. "Cavitation in Hydraulic Structures", Traducido al Ingles por V. Michels para J. Barry Cooke, Inc.
9. Hopping P.N., Mass, G. R., 1987. "Cavitation Damage on the Karun Dam", ACI Journal.
10. ICOLD, 1992. Committee of Hydraulics for Dams, "Spillways, Shockwaves and Air Entrainment", Bulletin 81.
11. Kenn, M., and Garrod, A., 1981. "Cavitation damage and the Tarbela tunnel collapse of 1974", Proc. I.C.E.
12. Marcano A., 2001. Caruachi Project on the Caroni river, - "Key Hydraulic issues related to the Second Stage of River Diversion", Special Seminar SS2, "Key Hydraulic issues in Large Water Projects", XXIX IAHR World Congress, Beijing, China
13. Marcano A., et alia, 2004. "3D CFD modeling-investigation of potential vortex formation at the intakes of Caruachi Powerhouse", Hydraulics of Dams and River Structures, Yandastood & Attari (eds) Taylor and Francis, ISBN 90 58096327, Teheran, Iran

14. Marcano et alia, 2004. "Numerical unsteady model simulation during the sluice closure of Caruachi Dam", Hydraulics of Dams and River Structures, Yanndstood & Attari (Eds) Taylor and Francis, ISBN 90 58096327, Teheran, Iran
15. Marcano, A., 1982. Open Channel Spillways flows with particular reference to air entrainment, trabajo especial de grado como requisito parcial para obtener el titulo de Magíster en Ciencias de la Universidad de Strahclyde, UK.
16. Marcano, A., 1999."Plunge pool Development at Guri Spillway", XXI IAHR Congress, Graz, Austria.
17. Marcano, A., Castillejo, N., 1984. "Model-prototype comparison of Aerating devices of Guri Dam Spillway", Symposium on Scale Effects on Hydraulic Structures, IAHR, September.
18. Marcano, A., Patiño, A., 1989."Cavitation damage at Chute 3 Stage II of Guri Spillway and solution to the problem", Congresso Internacional sobre Casos e Acidentes em Sistemas Fluidos, Anais, Sao Paulo.
19. Matos, J., Quintela, A., Sanchez, M., Dolz, J., 2000. "Air entrainment and safety against Cavitation damage in stepped spillways over RCC dams", Proceedings of the International Workshop on Hydraulics of stepped spillways, Zurich.
20. Méndez, I., Salazar, E., Marcano, A., Sánchez, M., Egusquiza, E., Pomares, J., Niñerola, D., Dolz, J., 2000. "Proyecto Caruachi - Investigación Experimental sobre la Fuerza Descendente (Downpull) Durante las Maniobras de Cierre de Emergencia de los Conductos de Desvío", XIX Congreso Latinoamericano de Hidráulica.
21. Montilla, G., 2006. "Correlación CFD-Modelo Físico del Comportamiento Hidrodinámico de los Ductos de Desvío de la Central Hidroeléctrica Caruachi", CIMENICS`2006.
22. Montilla, G., 2009. "Correlation Cfd - Physical Model Of The Sluices Of River Diversion Of The Caruachi Hydropower Plant", 33rd IAHR 2009 Congress - Water Engineering for a Sustainable Environment.
23. Montilla, G., Martinez, E., 2009. "Tocoma Hydroelectric Project, Venezuela", Spotlight IAHR Hydraulic Structure Section, Newsletter Vol.2.
24. Patiño, A., 1985. "Ensayos de Aireación en el Canal 2-Etapa III del Aliviadero de Guri", V Jornadas Nacionales de Ingeniería Hidráulica, Sociedad Venezolana de Ingeniería Hidráulica.
25. Peterka, A., 1953. "The Effect of Entrained Air on cavitation Pitting", Proc. Minnesota International Hydraulics Convention, Sept.
26. Prusza, Z., Mantellini, T., Semenkov, V., 1982. "Remedial measures against Cavitation damage in Spillways", IAHR Congress, Moscow.
27. Ripken J., Dahlin, W., 1972. "Guri Hydroelectric Project-Report on Spillway Cavitation Damage", Project n° 132, St. Anthony Falls Hydraulic Laboratory, Feb.
28. Roo, H. Editor, (en imprenta). "Memoria Técnica del Proyecto Guri"
29. Wei, C. Y. and De Fazio, F., 1982. "Simulation of free jet trajectories for the design of aeration devices for hydraulic structures", 4th International Conference on Finite Elements in Water Resources, Hannover, Republic of Germany.
30. Wood I.R 1985. "Air Water Flows", IAHR, 21st Congress, Melbourne, Australia.

31. Wood, I. R. 1991.. Editor, "Air Entrainment in Free surface flows", IAHR, Hydraulic Structures Design Manual", Balkema.
32. Zagustin, K, Castillejo, N., 1983. "Model-Prototype correlation for flow Aeration in the Guri Dam Spillway", XX IAHR Congress, Moscow.
33. Zagustin, K., Castillejo, N., Mantellini, T, 1982.

Appendix B.2

1. Rizzo, P. 2006. "2.0 Summary Description of the Taum Sauk Plant, R5 063551/06" Associates.
2. U.S. Society on Dams, 2013. "Dams of the United States – A Pictorial Display of Landmark Dams"
3. FERC Staff Report, 2006. "Taum Sauk Pumped Storage Project (No. P-2277)", Dam Breach Incident
4. Leonard, C. 2005. "Taum Sauk Reservoir fails", Associated Press.
5. Carrera, E., et alia, 1979, "Raising Guri Dam: Stage Construction to meet load Growth ", Q48, R14, XIII ICOLD Congress on Large dams, New Delhi,
6. Castillejo, N., et alia. 1985. "Raising the Spillway Flip Buckets of Guri Dam". Special Seminar of Hydraulic Structures, XXVI Congress of the International Association for Hydro environmental Engineering and Research ", IAHR, Lausanne, Switzerland
7. De Fries, A., et alia, "Raising of Guri Dam", 50th anniversary of LNEC, Lisbon Portugal
8. Donelly, R.,General Reporter Q97, (2015) "Spillways" , 35th ICOLD Congress, Stavanger
9. Harza Eng Company. 1981. "Revised Probable maximum Flood Analysis for Guri Dam"
10. Harza Engineering Company, 1976. Caroni River hydroelectric Development. Guri final Stage. Civil Design memoranda. Raising of the Existing Spillway.
11. ICOLD, 1992., Bulletin. "Raising of existing Dams".
12. ICOLD, 2004. "Design of System of weirs to protect The Llovizna falls", Korea
13. ICOLD, Q70, "Staged construction, raising or modifications of dams ", ICOLD Congress,
14. Marcano A., et alia. 1987. "Aeration at Guri Final Stage Spillway", Model-prototype Correlation of Hydraulic Structures, International Symposium, ASCE, Ed Philip Burgi.
15. Marcano, A., 1999, "Monitoring Guri Spillway Plunge Pool" XXVIII Congress of the International Association for Hydro environmental Engineering and Research, IAHR, Graz Austria.
16. Marcano, A., et alia. 1995. "Operation of Guri dam Spillway during raising of the Dam". ICOLD, XXII Congress of the International Commission of Large Dams, San Francisco, Q 63d
17. Pinto, N. 1994. "Deterioration of Spillway and Outlet Works". General Reporter Q 71. 18th ICOLD Congress, Durban

18. Prusza, Z. 1994. "Design and Construction measures to avoid cavitation in a Large Spillway". ICOLD 18th Congress, Durban, Q 71, R 15
19. Semenkov, V. 1979. "Large Capacity Outlet and Spillways". General reporter, Q 50, 13th, ICOLD. Congress, New Delhi

Appendix B.3

1. Córdova, J. Et al. 2006. "Estimación de hidrogramas de crecidas extremas" in book: Ingeniería Forense y Estudios de Sitio. Guía para la prevención de Gestión de Riesgos. Minipres C.A. pp. 57-73. Caracas. Venezuela
2. Díaz, L. 2015. "Relato sobre la ruptura de la represa de El Guapo, año 1999". Retrieved from http://historiaguapena.blogspot.com/2015/12/un-16-de-diciembre-se-revento-la-represa_14.html
3. Fiedler, W. 2016. "Managing Dam Safety risks Related to Hydraulic Structures". 6th International Symposium on Hydraulic Structures. Hydraulic Structures and Water System Management. Portland, Oregon USA.
4. Grases, J. et al. 2000. "Efectos de las Lluvias Caídas en Venezuela en Diciembre de 1999". CAF – PNUD, Caracas, Febrero 2000.
5. Hattingh, L. 2015. "Practical aspects of dam break analysis" at Dam Break Analysis Workshop. Pretoria. South Africa.
6. Isava, M. 1975. "Proyecto Embalse El Guapo, Estado Miranda". Instituto Nacional de Obras Sanitarias - INOS, Caracas.
7. Méndez, W. et al. 2018. "Collapse of the El Guamito Dam (Venezuela) to the Extraordinary Rains of December 1999: Hydro-geomorphological Conditioners in the Guapo River Basin". Anuário do Instituto de Geociências – UFRJ
8. Méndez, W. et al. 2008. "Geomorfología de la cuenca de drenaje del río Guapo, y sus implicaciones hidrológicas en el colapso de la presa "El Guamito" en Diciembre de 1999, estado Miranda, Venezuela". Universidad Pedagógica Experimental Libertador. I Jornadas de Gestión de Riesgos e Impacto Ambiental. Río Chico. Venezuela.
9. Ministerio de Obras Públicas. 1976. "Grandes Presas de Venezuela". Dirección General de Recursos Hidráulicos. Revista El Agua – Número Especial – Grandes Presas de Venezuela, Caracas.
10. Rotundo, L. 2000. "El Guapo. ¿Por qué falló?" Vertiente. HIDROCAPITAL. Pp. 14-16 N° 3. Caracas. Venezuela.
11. Stolpa, D. 2018. "Una Introducción a Análisis de Suficiencia de Vertederos para Presas". Universidad Católica Boliviana, "San Pablo" de Tarija. Bolivia
12. Suárez, L, et al. 2006. "Overtopping of Two Dams in Venezuela" In International Symposium on Hydraulic Structures. Recent developments on hydraulic structures. From hybrid modeling to operation and repairs. Ciudad Guayana. Venezuela
13. Suárez, L, et al. 2016. "Lecciones aprendidas de los Incidentes y fallas en la Presas de Venezuela"
14. US Army Corps of Engineers. 2017. "Overtopping of Walls and Stilling Basin Failure". Best Practices in Dam and Levee Safety Risk Analysis. Part F – Hydraulics Structures. Chapter F-2.

Appendix B.4

1. “Appendix 1: Niobrara River Background" (PDF). Niobrara River Instream Flow Documents. Nebraska Department of Natural Resources.
2. ASDSO, “Spencer Dam Filure Investigation Report” 2020. Association of State Dam Safety Officials.
3. Constantinescu, George; Garcia, Marcelo; Hanes, Dan, eds. 2016. River Flow 2016: Proceedings of the International Conference on Fluvial Hydraulics, St. Louis, USA. CRC Press. pp. 1448–1449. ISBN 1-31564-447-9.
4. Dockendorf, Randy 2019. "Spencer Dam Break Causes Corps To Raise Gavins Point Dam Releases". Yankton Daily Press & Dakotan.
5. Gutzmer, Michael P.; King, Justin W.; Overhue, David P. 1996. "Environmental Impacts in the Vicinity of Spencer Hydropower Dam During Sluicing Activities in the Niobrara River, Nebraska" (PDF). Transactions of the Nebraska Academy of Sciences and Affiliated Societies. 81.
6. Hammel, Paul 2019 "'I've never seen anything close to this: Rancher describes harrowing evacuation near collapsed Spencer Dam". Omaha World-Herald.
7. Hammel, Paul 2019. "Spencer Dam's collapse raises questions about what went wrong with 92-year-old structure". Omaha World-Herald.
8. Hendee, David 2015. "NPPD to shut down Spencer Dam hydropower plant, give up water rights on Niobrara River". Omaha World-Herald.
9. LEX182019. “Historic Flooding Causes Nebraska Dam Failure,”
10. NWSMBRFC tweet on 14 March 2019 with map of the flooding downriver to the Missouri River upon failure, by National Weather Service Missouri Basin River Forecast Center
11. Salter, Peter 2019. "11-foot wall of water: One dam breaks, three counties suffer". Lincoln Journal Star. Retrieved
12. Shultz, Steven 2010. "The Extent and Value of Agricultural, Municipal and Industrial Water Use in the Niobrara Basin" (PDF). Nebraska Game and Parks Commission.
13. Strange Sounds -Mar 16, 2019 “ Deadly ice jam destroys Spencer Dam and nearby highway – Cattle stranded on small inlets in Platte River in rural Nebraska,”
14. The Journal of Gazette, Associated Press, 2019. “Probe: Aging US dams pose risks,”
15. Warneke, Kent; Guenther, Jerry 2019. "Spencer dam breaks, many towns evacuated". Norfolk Daily News.
16. World-Herald Bureau, March, 2019. “Spencer Dam collapse may be first in nation caused by giant ice chunks,”
17. World-Herald News Service 2015. “NPPD to shut down Spencer Dam hydropower plant,”
18. Yoders, Jeff 2019. "Nebraska Flood Exposes Weak Points". Engineering News-Record.

Appendix B.5

1. Singhal, S. (2017) Comprehensive rehabilitation repair of Maneri dam in Distt. Uttarkashi, Third National Dam Safety Conference. Roorkee 2017.
2. Indian Standard Code: 10262-2009
3. Indian Standard Code: 456-2000
4. Indian Standard Code: 383-2016
5. Indian Standard Code: 269-2015
6. Indian Standard Code: 2386-1963

Appendix C

1. Abdelnour, R., et alia, 2003. "Development of Boom design Procedure and Site Selection to Improve The Warning and the Efficiency of Hydro plants", ICOLD Montreal, Canada.
2. Beattie, J.F. et al., Lift and Drag Forces on a Submerged Circular Cylinder. Paper presented at the Offshore Technology Conference, Houston, Texas, April 1971. Paper Number: OTC-1358-MS
3. Crissman, R. D., Abdelnour, R. and Shen, H. T., Design Alternatives for the Lake Erie-Niagara Ice Boom, 8th Workshop on River Ice, Kamloops, BC, Canada, 1995
4. Downing-Kunz M., et al, 2011. "Flow-induced forces on free-floating macrophytes", Hydrobyologia, 671:121–135- DOI 10 10.1007/s10750-011-0709-1, Springer Science Business Media B.V.
5. FERC, 1992. "Guidelines for Public Safety at Hydropower Projects", Division of Dam safety and Inspections, FERC.
6. Hamilton, H. L. The Lake Erie-Niagara River Ice Boom. Operations and Impacts, Niagara River Ice Boom Investigations, November 1983.
7. Morse, B. 2000. "Factors Affecting Ice Loads on Wide Structures", National research Council Canada.
8. Perham, R. E., Floating Debris Control: A Lirerature Review, U.S. Army Corps of Engineers, Technical Report, REMR-HY-2, June 1987.
9. Perham, R. E., Elements of Floating-Debris Control Systems, U.S. Army Corps of Engineers, Technical Report, REMR-HY-3, September 1988.
10. Razek Abdelnour, 2001. "Ice Booms in Rivers; Lessons Learned and the Development of Reliable Solutions", Fleet Technology Limited, Ontario Canada.
11. Razek Abdelnour, Andrew Liddiard and Stéphane Dumont 2002. "Cost effective alternative for sinking and re-floating ice booms", International Association of Hydraulic Engineering and Research.
12. Razek Abdelnour, Corby Nicholson and Yuxiang Gong 1998. "The design, fabrication and deployment of an ice boom to protect two hydroelectric power plant water intakes from ice blockages during winter". Fleet Technology Limited, Canada.

13. Razek Abdelnour, Elie Abdelnour, Etienne Robert, Jarrod Malenchak, 2017. "Ice Boom Design Challenges in a Very Large, High-Latitude, North-Flowing River", 19th Workshop on the Hydraulics of Ice Covered Rivers.
14. Urroz G., Ettema R. (1987) Simple-shear box experiments with floating ice rubble. Cold Reg. Sci. Technol., 14 (2), pp. 185-199
15. Valbuena, J., et alia, 2010. "Optimization of the System of Caruachi Dam Safety Floating Barriers", Hydropower and Dams.
16. Wahl, T., et al, "Investigations of Debris and safety Boom Alternatives for Bureau of Reclamation
17. Wahl, T. L., Investigation Of Debris And Safety Boom Alternatives For Bureau Of Reclamation USA, Technical Report, R-92-04, February 1992.
18. Wallerstein, N. and Thorne, C. R., Debris Control at Hydraulic Structures in Selected Areas of the United States and Europe, US Army Corps og Engineers, Control Report CHL-97-4, December 1997.
19. World Meteorological Organization (WMO), Global Guide to Tropical Cyclone Forecast, WMO-No 1194, 2017

Appendix D

Climate Change

1. D. Winstantley and S. A. Changnon, Insights to Key Questions about Climate Change, IllinoisState Water Survey, April 2004.
2. M. d. S. Mesquita, V. Veldore, L. Li, R. Krishnan, Y. Orsolini, R. Senan, M. V. S. Ramarao, and E. Viste, "Forecasting India's Water Future: The NORINDIA Project Sheds Light on How Climate Change Could affect Monsoons, Droughts, and Glaciers in Northern India." EOS. Earth and Space Science News, May 15, 2016.
3. L. Strelich, "Improving Indian Summer Monsoon Prediction," EOS Earth and Space ScienceNews, January 15, 2016.
4. C. C. Raible, D. Stephenson, and G. Leoncini, "Predicting the Risk and Occurrence of Extratropical Cyclones," 5th Workshop on European Storms: Impacts and Predictability, September 2015.
5. Emily Underwood, "How Global Warming's Effect on Clouds May Make It Rain Harder,"EOS Earth and Space Science News, April 2017.
6. M. McGrath, "Climate Change: Last Decade Confirms as Warmest on Record," BBC, January15, 2020.
7. M. McGrath, "Cyclone Idai: What's the Role of Climate Change?" BBC, March 20, 2019.
8. T. Wahl, P. J. Ward, H. C. Winsemius, A. A. Kouchak, J. Bender, I. D. Haigh, S. Jain, M. Leonard, T. I. E. Veldkamp, and S. Westra, "When Environmental Forces Collide," EOS Earthand Space Science News, March 2019.
9. J. Gunderson, "Climate-Change Fears Drive Global Projects: Governments Prepare for WaterScarcity and More Floods," Engineering News Records, December 10, 2012.
10. J. Duncombe, "Living in Fear of Rain," EOS Earth and Space Science News, November2019.

11. D. Tullos and S. K. Jain, "Addressing Challenges of Mitigating Flood Risk in Mountain Areas- Workshop on Modeling and Managing Flood Risk in Mountainous Areas, Folsom, California, February 2015," EOS Earth and Space Science News, July 15, 2015.
12. S. Stanley, "How Will Melting Glaciers Affect Steamflow?," EOS Earth and Space ScienceNews, April 2019
13. Earth Institute, "Con Ed Study Details Projected Climate Impacts on Energy Systems, "Columbia University, December 20, 2019.
14. BBC, "Venice Floods: Climate Change Behind Biggest Tide in 50 Years, Says Mayor," BBC, November 13, 2019.
15. N. Berry, "Is Climate Change Behind Venice Flooding?" BBC, November 13, 2019.
16. NHK, "Researchers: Global Warming May Slow Typhoons," NHK, January 9, 2020.
17. R. E. Benestad, K. M. Parding, A. Mezghani, A. Dobler, O.A. Landgren, H. B. Erlandsen, J. Lutz, and J. E. Haugen, "Stress Testing for Climate Impacts with Synthetic Storms," EOS Earth and Space Science News, January 14, 2019.
18. D. P. Lettenmaier, G. McCabe, and E. Z. Stakhiv, "Global Climate Change: Effect on Hydrologic Cycle," Water Resources Handbook (Chapter 29), McGraw-Hill, 1996.
19. J. Kluger and H. Sweetland, "The Storms Keep Getting Stronger. And So Do Us. Learning from Disaster," Time (Magazine), September 25, 2017.
20. C. W. Shabica, the Illinois Lakeshore 2100, Adaptive Shorelines to Cope with ClimateChange, Winnetka Historical Society Gazette, Fall/Winter 2019.
21. L. Sahagun, "Extreme Storm could Overwhelm Southern California Dam and FloodThousands," Los Angeles Times, November 6, 2019.
22. W. R. Fiedler, "Managing Dam Safety Risks Related to Hydraulic Structures," Hydraulic Structures and Water System Management, 6th IAHR International Symposium on Hydraulic Structures, Portland, June 2016.
23. ICOLD, World declaration on Dam Safety, International Commission on Large Dams, October 18, 2019.
24. Water Power, "ICOLD gets Technical about Climate Change," International Water Power &Dam Construction, March 2019.
25. USBR, Secure Water Act Section 9503(c) - Reclamation Climate Change and Water 2016, USBR, 2016.
26. USBR and USACE, Addressing Climate Change in Long-Term Water Resources Planning and Management, USBR, January 2011.
27. M. A. Hariri-Ardebili and L. K. Nuss, "Next Generation of Dam Safety and Security Frameworks: A Big Picture," USSD Dams and Levees, Issue No. 179, Fall 2019, Bulletin of the United States Society on Dams, 2019.
28. ASDSO, Living with Dams: Extreme Rainfall Events, Association of State Dam SafetyOfficials, 2015.
29. J. Wilson, "A Race against Time, America" s Flood Risk from Aging Dams," proceedings, 36th Annual USSD Conference, Denver, 2016.

Hazard Classification

30. USBR, Downstream Hazard Classification Guideline, Technical Memorandum No. 11, USBR, 1988.

Hydraulic Modeling

31. ICOLD, Technical Advancements in Spillway Design - Progress and Innovations from 1985 to2015, September 2016 Bulletin, ICOLD, 2016.
32. P. Novak, V. Guinot, A. Jeffrey, and D. E. Reeve, Hydraulic Modelling – an Introduction: Principles, Methods and Applications, Spon Press, 2010.
33. J. A. Roberson and C. T. Crowe, Engineering Fluid Mechanics, Houghton Mifflin Co., 1975.
34. USACE, “Hydraulic Design Criteria,” USACE, 1987. Physical Hydraulic Modeling
35. S. Smith McEwen, “History Lesson - Restoration Required: The Mississippi River Basin Model,” Civil Engineering Magazine, March 2020.
36. A. Manning, Gross Dam Spillway Design Being Put to the Test by CSU Civil Engineers: Raising a Dam, Colorado State University, September 24, 2019.
37. UWRL, Oroville Dam 1:50 Scale Model at the UWRL Helps Test Repair Plans BeforeConstruction, Utah Water Research Laboratory, 2017.
38. M. Weiser, Oroville Dam in Miniature: Scale Model Helps Repair Damaged Spillway, WaterDeeply, May 29, 2017.
39. B. McCombs, Model for Oroville Dam Repair is in Utah Warehouse, the Associated Press, and June 26, 2017.
40. D. Brekkle, Building a Model to Help Restore Oroville Dam’s Shattered Spillway, KQED,June 30, 2017.
41. R. A. Davidson, Bonneville Second Powerhouse ERDC Turbine Model Operating Range Investigation, ERDC/CHL TR-18-13, CHL, USACE, August 2018.
42. R. A. Davidson, Bonneville First Powerhouse ERDC Turbine Model Operating Range Investigation, ERDC/CHL TR-17-9, CHL, USACE, May 2017.
43. M. J. Briggs, Basics of Physical Modeling in Coastal and Hydraulic Engineering, ERDC/CHL CHETN-XIII-3, ERDC, USACE, September 2013.
44. ASCE, Hydraulic Modeling – Concept and Practice, ASCE Manuals and Reports onEngineering Practice No. 97, ASCE, 2000.
45. USBR, Water Measurement Manual, USBR, 2001.
46. F. Şentürk, “Model Techniques Used for Solving Special Flow Problems, “Hydraulics ofDams and Reservoirs (Chapter 13), Water Resources Publications, 1994.
47. S. A. Hughes, Physical Models and Laboratory Techniques in Coastal Engineering, Advanced Series on Ocean Engineering – Vol. 7, World Scientific Publishing Co. Pte. Ltd., 1993.
48. P. Novak, A.I.B. Moffat, C. Nalluri, and R. Rarayanan, “Scale Models in HydraulicEngineering,” Hydraulic Structures (Chapter 15), Unwin Hyman Ltd., 1990.
49. IAHR, Directory of Hydraulic Research Institutes and Laboratories, IAHR and DELFT, 1987.

50. A. Chadwick and J. Morfett, "Dimensional Analysis and the Theory of Physical Models,"Hydraulics in Civil Engineering (Chapter 13), Allen & Unwin, 1986.
51. Hunter Rouse and S. Ince, History of Hydraulics, Iowa Institute of Hydraulic Research, 1980.
52. M. S. Yalin, Theory of Hydraulic Models, the Mcmillan Press Ltd., 1971.
53. P.C. Pethe, M.R. Kulkarni, V.V.Bhosekar and P.B. Deolalikar, "Hydraulic Model Studies for Tala Dam Spillway, Bhutan," CWPRS, ISH Journal of Hydraulic Engineering, June 2012.
54. Ahmad, Z. (2018). Physical Model Study of Spilling and Energy Dissipation Arrangements of Malana Dam, Kullu, India. Daniel Bung, Blake Tullis, 7th IAHR International Symposium on Hydraulic Structures, Aachen, Germany, 15-18 May. doi: 10.15142/T35934 (978-0-692-13277-7).
55. Ahmad, Z. (2011). Physical modeling of desilting chamber, 2nd Sediment Management Conference, The Grand, Vasant Kunj, New Delhi, November 11, 2011.
56. Ahmad, Z. (2010). Physical model study for intake and desilting chamber of Nafra Hydro Electric Project, Sew Nafra Power Corporation Private Limited, 6-3-871, Snehalata, Greenlands Road, Begumet, Hyderabad.
57. Ahmad, Z. (2010). Physical model study for Spillway of NAFRA Hydro Electric Project, Sew Nafra Power Corporation Private Limited, 6-3-871, Snehalata, Greenlands Road, Begumet, Hyderabad.
58. Chanson, H. (2004). The Hydraulics of Open Channel Flow: An Introduction. Elsevier, New York.
59. Noval, P., Guinot, V., Jeffrey, A., and Reeve, D.E., (2010). Hydraulic Modelling - an Introduction : Priniciples, Methods and Applications. CRC Press, New York.
60. French, R.H. (1985). Open Channel Hydraulics, McGraw-Hill, New York.
61. Hussain, S., Hussain, A., Ahmad, Z. (2013). "Discharge characteristics of orifice type spillways." Hydro International -2013 conference, IIT Madras, 835-843.
62. Hussain, S., Hussain, A. and Ahmad, Z. (2014). "Discharge characteristics of orifice spillway under oblique approach flow. " J. Flow Measurement and Instrumentation, Elsevier, Cambridge, United Kingdom, 39, 9-18.
63. IS 12804:2004. Criteria for estimation of aeration demand for spillway and outlet structures. BIS, Manak Bhavan, New Delhi.
64. IS 6934:1998. Hydraulic design of high ogee overflow spillways-recommendations. BIS, Manak Bhavan, New Delhi.
65. Peterka, A.J. (1984). Hydraulic Design of Stilling Basins and Energy Dissipators. Eng. Monograph No. 25, USBR.
66. USBR (2011). Design of small dams. A water Resources Technical Publication, USBR.
67. Wood, I.R. (1991). Air Entrainment in Free-Surface Flows, A.A. Balkema, Rottordam

Numerical Modeling

68. C.L. Rakowski, M.C. Richmond, J.A. Serkowski, and P.D.J. Romero-Gomez, BonnevilleProject: CFD of the Spillway Tailrace, PNNL-21842, Pacific Northwest National Laboratory, November 2012.

69. C.L. Rakowski, M.C. Richmond, and J.A. Serkowski, Technical Review of the CENWP Computational Fluid Dynamics Model of the John Day Dam Forebay, PNNL-20096, Pacific Northwest National Laboratory, 2010.

70. C.L. Rakowski, W.A. Perkins, M.C. Richmond, and J.A. Serkowski, Computational Fluid Dynamics Modeling of the John Day Dam Tailrace, PNNL-19349, Pacific Northwest National Laboratory, July 2010.

71. C.L. Rakowski, M.C. Richmond, J.A. Serkowski, and W.A. Perkins, Computational Fluid Dynamics Modeling of the Bonneville Project: Tailrace Spill Patterns for Low Flows and Corner Collector Smolt Egress, PNNL-20056, Pacific Northwest National Laboratory, 2010.

72. C.L. Rakowski, M.C. Richmond, J.A. Serkowski, and W.A. Perkins, Determining Columbia and Snake River Project Tailrace and Fore bay Zones of Hydraulic Influence Using MASS2. Modeling, PNNL-20030, Pacific Northwest National Laboratory, 2010.

73. C.L. Rakowski, W.A. Perkins, M.C. Richmond, and J.A. Serkowski , Simulations of The Dalles Dam Proposed Full Length Spillwall - Memoradum for Record , PNNL-17322, Pacific Northwest National Laboratory, February 2008.

74. C.L. Rakowski, M.C. Richmond, and J.A. Serkowski, Forebay Computational Fluid Dynamics Modeling for The Dalles Dam to Support Vortex Suppression Device Studies - Memorandum for Record, PNNL-16121, Pacific Northwest National Laboratory, Sept. 2006.

75. C.L. Rakowski, and M.C. Richmond, John Day Tailrace MASS2 Hydraulic Modeling – Memorandum for Record, PNNL-14491, Pacific Northwest National Laboratory, June 2003.

76. L.L. Ebner, R. Lee, S.K. Askelson and P. Williams, "Hydraulic Models, Once You Have It –Exercise It," CENWP-EC-HD, USACE, 2002.

77. L.L. Ebner, S.K. Askelson1, E.A. Thompson and N.C. Cox, "Numerical Modeling of the Spillways for the Dam Raise at Isabella Dam," proceedings, 6th International Symposium on Hydraulic Structures, Portland, Oregon, June 2016.

78. J. Burnham, Modeling Dams with Computational Fluid Dynamics: Past Success and NewDirections, Flow Science Inc., 2011.

79. P.G. Chanel, an Evaluation of Computational Fluid Dynamics for Spillway Modeling, M.S.Thesis, University of Manitoba, 2008.

80. P.G. Chanel and J.C. Doering , "An Evaluation of Computational Fluid Dynamics for Spillway Modelling," proceedings, 16th Australasian Fluid Mechanics Conference, Australia, December 2007.

81. S. Heimerl, M. Hagmeyer, and C. Echteler, "Numerical Flow Simulation of Pool-Type Fish ways: New Ways with Well-Known Tools," Hydrobiology, Springer, 2008.

82. D. Gessler, and B. Rasmussen, "Testing the Water with CFD," International Water Power &Dam Construction, Water Power Magazine, January 2006.

83. DKH Ho and K.M. Riddette, "Application of Computational Fluid Dynamics to Evaluate Hydraulic Performance of Spillways in Australia," Australian Journal of Civil Engineering, Vol. 6, No. 1, Institution of Engineers Australia, 2010.

84. A. Ruprecht, T. Helmrich, T. Aschenbrenner, and T. Scherer, "Simulation of Pressure Surge in a Hydro Power Plant Caused by an Elbow Draft Tube," University of Stuttgart , 2001.

85. F. Ye, Y. J. Zhang, H. V. Wang, M. A.M. Friedrichs, I. D. Irby, E. Alteljevich, A. Valle-Levinson, Z. Wang, H. Huang, J. Shen, and J. Du, "A 3D Unstructured-Grid Model for Chesapeake Bay: Importance of Bathymetry," Ocean Modeling, Vol. 127, Elsevier, 2018.

86. S. Heimerl, M. Hagmeyer, and B. Kohler "En Route with Water Particles in Fishways – How to Explain Flow Structure in a Pool-Type Fishway, proceedings, Hydro 2005 on Policy into Practice, Villach, 2005.

87. J. V. Letter, Jr., N. K. Raphelt, and S. H. Scott, Designing the River of the Future – US ArmyEngineer ERDC Numerical Modeling Techniques, a presentation, USACE, 2008.

88. ASCE, Emerging Computer Techniques in Stormwater and Flood Management-ConferenceProceedings, ASCE, November 1983.

89. J. P. Holland, "Advancements in Computational System for Hydraulic and Hydrologic Modeling," Issues and Directions in Hydraulics, proceedings, 1996 Iowa Hydraulic Colloquium - Iowa City, Nakato and Ettema (editors), A. A. Balkema, 1996.

90. B. C. Yen, "Hydraulics for Excess Water Management," Water Resources Handbook-Chapter25, Larry W. Mays (Editor in Chief), McGraw-Hill, 1996.

91. E. B. Wylie, V. L. Streeter and L. Suo, Fluid Transients in Systems, Prentice Hall, 1993.

92. M. H. Chaudhry, Applied Hydraulic Transients, Van NostrandReinhld Co., 1987.

93. U. S. Dixit, Finite Element Method: an Introduction, Indian Institute of Technology, 2011.

94. J. N. Reddy, Solutions Manual for an Introduction to the Finite Element Method, McGraw- Hill, 2005.

95. E. F. Brater, H. W. King, J. E. Lindell and C.Y. Wei, "Computer Programs in Hydraulics,"King's Handbook of Hydraulics, Seventh Edition (Section 14), the McGraw-Hill Co., 1996.

96. D. Ford and D. Hamilton, "Computer Models for Water-Excess Management," WaterResources Handbook (Chapter 28), L. W. Mays (Editor), McGraw-Hill, 1996.

97. R. B. Ambrose, Jr., T. O. Barnwell, Jr., S. C. McCutcheon, and J. R. William, "Computer Models for Water-Quality Analysis," Water Resources Handbook (Chapter14), L. W. Mays (Editor), McGraw-Hill, 1996.

98. Computational Fluid Dynamics Task Committee, Computational Fluid Dynamics: Applications in Water, Wastewater, and Storm water Treatment, ASCE, 2019.

99. E. F. Brater, H. W. King, J. E. Lindell and C.Y. Wei, "Computational Hydraulics," King'sHandbook of Hydraulics, Seventh Edition (Section 13), the McGraw-Hill Co., 1996.

100. Task Committee on Software Evaluation, Guide for Evaluating Engineering Software, ASCE Technical Council on Computer Practices, ASCE, 1989.

101. WES, CE-QUAL-R1: A Numerical One-Dimensional Model of Reservoir Water Quality: User's Manual, Instruction Report E-82-1, Waterways Experiment Station, USACE, 1995.

102. Environmental and Hydraulic Laboratories, CE-QUAL-W2: A Numerical Two-Dimensional, Laterally Averaged Model of Hydrodynamics and Water Quality: User's Manual, Instruction Report E-86-5, Waterways Experiment Station, USACE, 1986.

103. D. M. Gee, Guideline for the Calibration and Application of Computer Program HEC-6, Hydrologic Engineering Center, USACE, October 1982.

104. HEC, Water Quality for River-Reservoir Systems, the Hydrologic Engineering Center, USACE, 1978.

105. D. M. Gee, Role of Calibration in the Application of HEC-6, Technical Paper No. 102, Hydrologic Engineering Center, USACE, October 1982.

106. A. M. Proudvsky and V. P. Seredavkin, "Combined Application of Physical and Numerical Modeling in Hydraulic Research," Recent Developments on Hydraulic Structures – from Hybrid Modeling to Operation and Repairs, proceedings, International Symposium on Hydraulic Structures, Venezuela, Editor A Marcano, et alia, October 2006.

107. S. L. Gorbachev, V.A. Maksimovich, V.O. Saranchev, and V.M. Semenkov, "Hybrid Simulation of Vortex Formation at Intake," Recent Developments on Hydraulic Structures - from Hybrid Modeling to Operation and Repairs, proceedings, International Symposium on Hydraulic Structures, Venezuela, Editor A Marcano, October 2006.

108. S. Erpicum, B.J. Dewals, P. Archambeau S., and M. Pirotton, "Composite Hydraulic Modeling to Design a Complex Dam Spillway," proceedings, 36th IAHR World Congress, Hague, 2015.

109. S. Erpicum, P. Archambeau, B.J. Dewals, S. Detrembleur, A. Lejeune. and M. Pirotton, "Interactions between Numerical and Physical Modeling for the Design and Optimization of Hydraulic Structures - Example of a Large Hydroelectric Complex,", Recent Developments on Hydraulic Structures - from Hybrid Modeling to Operation and Repairs, proceedings, International Symposium on Hydraulic Structures, Venezuela, Editor A Marcano ,October 2006.

110. G. Nikhil B., "Physical and Numerical Modeling of an Orifice Spillway," International Journalof Mechanical and Production Engineering, Vol. 3, Issue 10, October 2015.

111. O. Herrera-Granados and S. W. Kostecki, "Numerical and Physical Modeling of Water Flow over the Ogee Weir of the New Niedów Barrage," Journal of Hydrology and Hydromechanics-64, 2016.

112. Z. Nisar, M. K. Sarwar, and G. Nabi, "Hydraulic Performance Assessment of an Orifice Spillway using CFD Modeling," Research Paper, University of Engineering and Technology- Lahore, Pakistan, 2015.

113. M. Rafi, A. Ali, G. Qadir, and R. Ali, "Modeling the Mangla Dam Spillway for Cavitation andAerators Optimization," Journal of Water Resource and Protection-4, SciRes, 2012.

114. B. Margeirsson, Computational Modeling of Flow Over a Spillway in Vatnsfellsstifla Dam in Iceland, M.S. Thesis, Chalmers University of Technology, Sweden, 2007.

115. C.Y. Wei and F. G. DeFazio, "Simulation of Free Jet Trajectories for the Design of Aeration Devices on Hydraulics Structures," proceedings, Fourth International Conference on Finite Elements in Water Resources, Hannover, June 1982.

116. C.Y. Wei, and W. Y.J. Shieh, "Transient Seepage Analysis of Guri Dam," Journal of theTechnical Councils, ASCE, Vol. 105, April 1979.

117. C.Y. Wei and R. W. Jeppson, Finite Difference Solutions of Axisymmetric Infiltration ThroughPartially Saturated Porous Media, PRWG59c-6, UWRL, 1971.

118. R. W. Jeppson, Inverse Formulation and Finite Difference Solution for Flow from a Circular Orifice, Journal of Fluid Mechanics, Volume 40, Issue 114, January 1970.

119. A. L. Davis and R. W. Jeppson, Solving Three-Dimensional Potential Flow Problems byMeans of an Inverse Formulation and Finite Differences, UWRL, January 1973.

120. R. W. Jeppson, Simulation of Steady and Unsteady Flows in Channels and Rivers, UWRL, 1974.

121. R. W. Jeppson, "Inverse Formulation and Finite Difference Solution for Flow from a Circular Orifice." Journal of Fluid Mechanics, Volume 40, Issue 114, January 1970.

122. R. W. Jeppson, Solutions to Axisymmetric Seepage from Ponds through Homogeneous andNonhomogeneous Porous Media, UWRL, January 1968.

123. D. Fissel, R.ck Birch and J. Jiang, "Three Dimensional Computational Flow Modeling and High Resolution Flow Surveys for Fisheries Environmental Studies on the Upper Columbia River," proceedings, 2002 Hydro Vision Conference, Portland, July 2002.

124. J. Bountry, Y. Lai, and T. Randle, "Comparison of Numerical Hydraulic Models Applied to the Removal of Savage Rapids Dam Near Grants Pass, Oregon," proceedings, Eighth Federal Interagency Sedimentation Conference, Reno, April 2006.

125. R. Smith, The Evolution of the Commercial Computational Fluid Dynamics (CFD), Symscape, January 2008.

126. E. Usta, Numerical Investigation of Hydraulic Characteristics of Laleli Dam Spillway and Comparison with Physical Model Study, M.S. Thesis, Middle East Technical University, Ankara, and May 2014.

127. A. P. Singh, F. Bhat, and E. Gupta, "3-D Spillway Simulations of Ratle HEP (J&K) for the Design Alternatives to be Tested in Model Studies," proceedings, Symposium on Computational Fluid Dynamics Application in Water Engineering, Bangalore, June 2013.

128. N. Jaiayeri, J. Wendelbo, J. Groeneveld, A. J. Bearlin, and J. Gulliver, "Boundary Dam Total Dissolved Gas Analysis Using a CFD Model," proceedings, 37th Annual USSD Conference, USSD, April 2017.

129. J. Werner, J. H. Rutherford, K. Pate, D. Kirschbaum, J. Groeneveld, K. Moen, and J. Arnold,"Boundary Dam Spillway No. 2 Modifications," Newsletter No. 169, USSD, July 2016.

130. E. N. Dombeck, "Applications of FLOW-3D for Stability Analyses of Concrete Spillways atFERC Projects," proceedings, 36th USSD Annual Conference, Denver, April 2016.

131. N. Jalayeri, "Mitigating Total Dissolved Gas, "Flow Science, January 2020.

132. B. I. Ogor, S. Dietze, and A. Ruprecht, "Numerical Simulation of the Flow in Turbine-99Draft Tube," proceedings, third IAHR/ERCOFTAC Workshop on Draft Tube Flow, Sweden, December 2005.

133. A. Ruprecht, T. Helmrich, T. Aschenbrenner, and T. Scherer, "Simulation of Vortex Rope in a Turbine Draft Tube," proceedings, Hydraulic Machinery and Systems, 21st IAHR Symposium, Lausanne, September 2002.

134. B. I. Ogor, S. Dietze, M. Hagmeyer, and A. Ruprecht, "Numerical Simulation of the Turbine-99 Draft Tube with FENFLOSS and STAR-CD," University of Stuttgart, 2005.

135. M. J. Cervantes, T. F. Engström, and L. H. Gustavsson, Turbine-99 III, Proceedings, third IAHR/ERCOFTAC Workshop on Draft Tube Flows, Lulea University of Technology, Sweden, 2005.

136. GHD, Computational Fluid Dynamics Modeling for Dams, www.ghd.com/dams, 2015.

137. C.L. Rakowski1, M.C. Richmond1, J.A. Serkowski1, and Laurie Ebner, "Three-Dimensional Simulation of Forebay and Turbine Intakes Flows for the Bonneville Project," proceedings, 2002 Hydro Vision, PNNL-SA-35954, Pacific Northwest National Laboratory, 2002.

138. A. Lipej, 2002."CFX Reliable for Development of Hydraulic Machines," CFD Review.

139. J. A. Roberson, J. J. Cassidy, and M. H. Chaudhry, Hydraulic Engineering, John Wiley & Sons, Inc., 1998.

140. Wikipedia, Finite Difference Method, Wikipedia, March 2020.

141. C. F. Cerco and M. R. Noel, the 2002 Chesapeake Bay Eutrophication Model, EPA 903-R-04-004, ERDC-WES, USACE, July 2004.

142. COMSOL, The Finite Element Method (FEM), COMSL, 2017.

143. Wikipedia, Volume-of-Fluid Method (VOF), Wikipedia, 2020.

144. F. Lan, R. Waddell, M. Zusi, and B. Grant, "Replacing Ruby Dam Outlet Uses Computational Fluid Dynamics to Model Energy Dissipation," proceedings, 36th USSD Annual Conference, Denver, April 2016.

145. Flow Science, FLOW-3D and Volume-of-Fluid (VOF), www.flow3d.com/industries/water- environmental/dams/, Flow Science, 2020.

146. VA Tech Hydro, CFD Analysis, VA Tech-Andritz AG, 2007.

147. J. L. Martin and S. C. McCutcheon, Hydrodynamics and Transport for Water QualityModeling, Lewis Publishers, CRC Press, Inc., 1999.

148. G. L. Morris and J. Fan, "Modeling of Sediment Transport and Deposition in Reservoirs, "Reservoir Sedimentation Handbook (Chapter 11), McGraw-Hill, 1998.

149. D. Ford and D. Hamilton, "Computer Models for Water-Excess Management," WaterResources Handbook (Chapter 28), McGraw-Hill, 1996.

150. R. A. Wurbs, Computer Models for Water Resources Planning and Management, IWR Report94-NDS-7, Institute for Water Resources, USACE, and July 1994.

151. J. J. DeVries and T.V. Hromadka II, "Computer Models for Surface Water," Handbook ofHydrology (Chapter 21), McGraw-Hill, 1993.

Reservoir & Sedimentation Simulation

152. A. Acharya, Experimental Study and Numerical Simulation of Flow and Sediment Transport around a Series of Spur Dikes, Ph.D. Dissertation, the University of Arizona, 2011.

153. Flow Science Inc., Sediment Transport Model, Flow Science Inc., 2020.

154. J. L. S. Lopez, Mathematical Modeling of Sediment Deposition in Reservoirs, HydrologyPaper No. 95, Colorado State University, July 1978.

155. Subcommittee on Reservoir Sedimentation, Modeling Sediment Movement in Reservoirs, USSD Committee on Hydraulics of Dams, USSD, June 2015.

156. G. L. Morris and J. Fan, "Modeling of Sediment Transport and Deposition in Reservoirs", Reservoir Sedimentation Handbook (Chapter 11), the McGraw-Hill Co., 1998.

157. E. L. Pemberton and J. M. Lara, Computing Degradation and Local Scour: Technical Guideline for Bureau of Reclamation, Engineering and Research Center, USBR, Denver, January 1984.

158. G. W. Annandale, Scour Technology: Mechanics and Engineering Practice, McGraw-Hill, 2006.

159. G. W. Annandale, R. P. Humphries, A. J. Rock, M. S. McCray, and R. L. Adams, "Quantifying Rock Scour Extent Using Physical Hydraulic Model Studies," proceedings, 36th Annual USSD Conference, Denver, April 2016.

160. A. O. Smith, "Dam Safety Assurance Project Update: Bluestone Dam," 2019 Spring Bulletin, USSD on Dams & Levees, 2019.

161. M. Schleifstein, Mid-Barataria Sediment Diversion Could Create, Save 47 Square Miles of Land Over 50 Years, the New Orleans Advocate, Nola.com, January 4, 2020.

162. C.Y. Wei, F. Sotiropoulus, J. W. Meldrim, and M. Kaltsouni, "Investigation of Lake Current and Temperature Structures for Site Selection of a Water Intake," Recent Developments on Hydraulic Structures - from Hybrid Modeling to Operation and Repairs, proceedings, International Symposium on Hydraulic Structures, Venezuela, Editor A Marcano, October 2006.

163. P. P. Gadgea, V. Jothiprakash and V. V. Bhosekar, "Hydraulic Design Considerations forOrifice Spillways," ISH Journal of Hydraulic Engineering, January 2018.

164. C.Y. Wei and P. F. Hamblin, "Reservoir Water Quality Simulation in Cold Regions" proceedings, Cold.

165. USACE, ―Hydraulic Computations and Modeling of Ice-Covered Rivers, ‖ Ice Engineering: Engineering and Design, Manual 1110-2-1612, USACE, 2002.

166. W. Crosby, Flood Inundation Mapping Cadre Process and Procedures by the USACE's Modeling, Mapping, and Consequence Production Center (MMC), USACE, 2018.

167. James C. Shaug and John C. Bruch Jr., ―Solution of a Free-Surface Boundary Value Problem using an Inverse Formulation and the Finite Element Method, ‖ Journal of Hydrology, 26, Elsevier Scientific Publishing Co., 1975. Micro Modeling

168. AREC, Environmental River Engineering Project on the Mississippi, Applied River Engineering Center, St. Louis District Corps of Engineers, 1995.

169. Robert Dale Davinroy Jr., Physical Sediment Modeling of the Mississippi River on a Micro Scale, Master of Science Thesis, University of Missouri-Rolla, 1994.

170. USPTO, Method and Apparatus for Micro Modeling the Sediment Transport Characteristics of a River, Patent No. 5,653,592, U.S. Patent and Trademark Office, August 5, 1997.

171. Rob Davinroy, ―River Replication, Civil Engineering, July 1999.

172. Robert D. Davinroy, Roger A. Gaines, David C. Gordon, Leonard L. Hopkins, David P. Barretta, Jasen L. Brown, Michael T. Rodgers, Edward Brauer, Ashley N. Cox, Tim Lauth, and Bradley J. Krischel, Hydraulic Sediment Response Modeling, Replication Accuracy to the 130 River, Technical Paper M53, U.S. Army Corps of Engineers, St. Louis and Memphis Districts August 2011.

173. Robert D. Davinroy, David C. Gordon, Robert D. Hetrick, Steven L. Redington, and Claude N. Strauser, ―River Restoration Measures in Four Secondary Channels of the Mississippi River, an Interagency Success Story,‖ Proceedings, Wetlands Engineering and River Restoration Conference, Denver, March 22-27, 1998.

174. Bradley J. Krischel, Ashley N. Cox, Robert D. Davinroy, Jasen L. Brown, Timothy J. Lauth, Jason J. Floyd, Adam M. Rockwell, Hydraulic Sediment Response Model Investigation - The Mouth of the Meramec River HSR Model Mississippi River, River Miles 165.00 – 156.00, Technical Report M68, USACE, St. Louis District, Hydrologic and Hydraulics Branch, Applied River Engineering Center, April, 2014.

175. David C. Gordon, Robert D. Davinroy, and James W. Austin, ―Small Scale Physical Sediment Transport Modeling Approach Used To Solve a Chronic Dredging Problem on the Atchafalaya River at Morgan City, Louisiana,‖ Proceedings, Sediment and Flow Modeling, Seventh Federal Interagency Sedimentation Conference, Reno, Nevada, March 25-29, 2001.

176. David C. Gordon, & Robert D. Davinroy, ―Bridge Abutment Erosion Problem Solved with a Small Scale Physical Sediment Transport Modeling Approach,‖ Proceedings, Sediment and Flow Modeling Seventh Federal Interagency Sedimentation Conference, Reno, Nevada, March 25-29, 2001.

177. Little River Research & Design, EMRIVER Catalog, Little River Research & Design, 2020.

178. SAFHL, Research and Facilities - St. Anthony Falls Hydraulic Laboratory, Laboratory Brochure, St. Anthony Falls Hydraulic Laboratory, May 1982.

179. Elaine D. Foss and Fred W. Blaisdell, Let's Demonstrate Hydraulic Phenomena: A Guide for the Operation and Use of the Portable Demonstration Channel and Models of Hydraulic Structures, USDA Agricultural Research Service, July 1960.

180. FHWA, ―Introduction to Highway Hydraulics,‖ Training Course FHWA-NHI-135065, 2020.

181. JBA Trust, Mini Flume Technical Factsheet, JBA Trust, 2020.

182. Echavez A., G. 1996, "Modelos Hidráulicos de Fondo Fijo y la Ingeniería experimental", Universidad autónoma de México consejo nacional de ciencias y tecnologías, Asociación mexicana de hidráulica.

183. Vergara S, M., 1993,"Técnicas de modelación hidráulica", Alfaomega.

Appendix E

1. Cassidy, J.J. 2000. "Gated spillways and other controlled release facilities and dam safety", ICOLD 20th Congress, Beijing, Q 79, General Reporter

2. Erbisti, Paulo, 2014. "Design of Hydraulic Gates". CRC Press. Taylor & francis Group.

3. Indian Standard – IS:4622, IS:4623, IS:7718, IS:12804, IS:10096

4. International Research Journal of Engineering and Technology (IRJET), 2019. "Distant Monitoring and Controlling of Gated Dams using PLC & SCADA" presented on IRJET, Volume 6, Issue-04.

5. Levin, J., Bullard G.., Bowles, D., 2003. "Spillway Gate Reliability in the Context of Overall Dam Failure Risk" presented at US Society on Dams Annual Lecture, Charleston, South Carolina

6. US Army Corps of Engineers 2020. (Engineering & Design) EM 1110-2-2702; January 2020 "Design of Spillway Tainer Gates"

7. USACE-VII-3, 2015. Seismic Failure of Spillway Radial Gates.