Manual for Assessing Structural Safety of Existing Dams

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DRIP
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: Bhavani Sagar Dam, Tamil Nadu, India.

सत्यमेव जयते

Government of India
Central Water Commission
Central Dam Safety Organization

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Government of India
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Disclaimer

Before undertaking any rehabilitation works in a dam it is necessary to carry out needful investigations and testing and to check the stability / structural safety of various structures for various possible conditions of loading like seismic condition, flood discharging condition etc. The Central Water Commission under the Dam Safety and Improvement Project has undertaken to prepare this *Manual for Assessing Structural Safety of Existing Dams* to provide necessary guidance for carrying out the above activities for existing dams. The design studies required will vary from dam to dam depending on the type of problems encountered. While every effort has been taken to incorporate all basic details as per the latest state of the art yet it is not possible to cover all the conditions/problems which may be faced in the field. CWC absolves itself from any responsibility in this regard and dam owners and others involved with the dam rehabilitation activity should use their discretion in implementing the guidelines contained in this Manual**.**

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MESSAGE

The water resources sector in India is one of the most important sectors which require special attention. Along with the ever increasing pressure of the population and rapid pace of urbanisation there is an exponential increase in water demand. Owing to the monsoon type of climate, almost the entire water supply to the country in the form of rainfall takes place in the limited span of four months or less. It, therefore becomes imperative to store water in reservoirs so that the same can be used throughout the year or sometimes even across the years. In view of climate change, dams and reservoirs will have to play an even more important role as mitigation and adaptation infrastructures by way of creation of adequate storages in order to satisfy the vital needs of water, energy and food.

In order to handle varied challenges, it is utmost essential that the existing water storage assets of more than 300 billion cubic meter remain in sound health and safe condition, and deliver all intended benefits i.e. water supply, irrigation, hydropower, flood and drought mitigation as long as possible. In the present time, constructing a new dam is very challenging, given the population density and intensive land use, as also a host of other factors. The sustainable dam safety management, therefore is essential to ensure water and food security. The safe operation of dams and reservoirs through latest dam safety concepts is the need of hour to comprehensively address the dam safety management.

There are many aspects involved in dam safety management and publication of technical guidelines and manuals is one of the important requirements which is being carried out under Dam Safety Rehabilitation and Improvement Project (DRIP). The ***Manual for Assessing Structural Safety of Existing Dams*** deals with various aspects related to review of structural safety of existing dams including stability during unusual/extreme events like floods and earthquakes. It has been prepared based on contemporary global practices with contributions and value addition from renowned international and national experts. This Manual can be used as an excellent reference material by our engineers/dam owners while carrying out comprehensive safety review for their dams.

(R. K. Jain)
Chairman
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New Delhi
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FOREWORD

Dams and reservoirs are critical infrastructures, whose failure may have catastrophic consequences with risk of fatalities and high economic losses. Dams are considered to be safe if risks are kept under control through appropriate measures. The dam safety management comprises of important components such as dam surveillance and monitoring, dam safety review, operation and maintenance and rehabilitation measures as required. In order to ensure dam safety, a dam needs to be designed and constructed in a manner that ensures that it remains safe under all conceivable load combinations and operational conditions. The risk can be minimised by a reliable surveillance system for early identification of any form of deterioration of the dam, and any unanticipated occurrences, and by a maintenance programme aimed at preventing such occurrences. However, it is not possible to eliminate all risks. Thus the field investigations and comprehensive dam safety review are important for planning and finalizing rehabilitation programmes as required.

Presently India ranks third globally having 5334 large dams in operation and 411 under construction. Also storage created by these structures renders reliable security for water, food, energy and mitigation of droughts and floods. In Indian context with ever increasing population and limited water resources, upkeep of these assets is very essential. The publication of ***Manual for Assessing Structural Safety of Existing Dams*** is a long pending need of engineers/dam owners in India which will provide necessary guidance in carrying out structural review for their dams.

Today about 200 large dams in India are more than 100 years old, and each year this number is increasing spirally. These dams along with other dams need a sound system for structural assessment in order to work out rehabilitation measures as required. Current world wide practices have been utilized in preparation of this ***Manual for Assessing Structural Safety of Existing Dams***.

I hope that professionals engaged in the operation and maintenance of dams will find this manual very useful in managing the safety of their dams. I thank Dr. A. K. Chopra, Johnson Professor of Structural Engineering, Emeritus, University of California, Berkeley and Mr. Larry K. Nuss, Former Expert, USBR, USA for their contributions 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 finalization of this Manual.

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

New Delhi
November 2020

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PREFACE

The joint effect of aging/lack of maintenance, outdated past design practices, rapid downstream development coupled with increase in extreme meteorological events demands a fully funded and staffed dam safety programs, as well as sufficient funding for dam design review and repairs. Dam inspection programs routinely find deficiencies in dams, but inspections alone are not a remedy for these deficiencies. Without design review, maintenance, repair and rehabilitation, a dam may not be able to serve its intended purpose and could be at significant risk of failure. Responsibility for keeping dams functioning properly lies with the dam owners. Delays in repairing unsafe dams increases the probability of disasters which can otherwise be prevented by timely action.

Design review and rehabilitation of large dams is required to counter various deficiencies which develop with time and also to correct inadequacies on account of revisions in various standards/ guidelines. Deficiencies that are caused primarily by the ageing of a dam include degradation caused due to weathering, wear & tear of equipment due to normal use or misuse, loss of serviceability with prolonged operation, damage from natural events including floods, earthquake or landslides, damage from vandalism and war etc.

Deficiencies also include changes in:

- Hydrologic and Seismic loading,
- The state-of-practice of the structural analysis of dams,
- Potential Failure Modes for the dam,
- Materials characterization and strengths,
- Construction methods from original design,
- Dam performance due to deterioration, seepage, clogged drains, cracks, displacement.

The Manual contains eight chapters viz. Overview, Hydrological Review and Flood Routing Studies, Free Board Aspects, Investigations, Surveillance and Performance Monitoring, Concrete and Masonry Dams, Earth and Rock fill dams and Appurtenant works.

In addition the Manual contains appendices on various design aspects including Dr. A. K. Chopra's simplified dynamic analysis procedures for both Non Overflow and Gated Overflow Sections of Gravity dams.

This Manual is intended for use by the engineers who are responsible for reviewing the safety of dams in order to plan, design & construct various rehabilitation works, as necessary. The purpose of this Manual is to present an overview of the latest practices for assessing the structural safety of existing dams and to give enough references for the practicing engineers in India.

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ABBREVIATIONS

Acronyms used in this publication are as follows:	
BIS	Bureau of Indian Standards
CDSO	Central Dam Safety Organisation
CWC	Central Water Commission
SDSO	State Dam Safety Organisation
O & M	Operation and Maintenance
DRIP	Dam Rehabilitation and Improvement Project
DSRP	Dam Safety Review Panel
DTM	Digital Terrain Model
EAP	Emergency Action Plan
FMIS	Flood Management Information System
GPS	Global Positioning System (uses GPRS for data transmission like browsing the web)
FRL	Full Reservoir Level
MWL	Maximum Water Level
MDDL	Minimum Draw Down Level
DSL	Dead Storage Level
RT-DAS	Real Time Data Acquisition system
SCADA	Supervisory Control and Data Acquisition
DDMS	Dam Deformation Monitoring System
EDA	Energy Dissipation Arrangement
HM works	Hydro-Mechanical works
DG set	Diesel Generator set
ADAS	Automated Data Acquisition System
RMU	Remote Monitoring Unit
PC	Personal Computer

Chapter 1. OVERVIEW

Dams are constructed for utilization of river waters for irrigation, flood-control, hydro-power development, domestic and municipal supplies etc. Over the last fifty years, India has invested substantially in dams and related infrastructures. At the time of independence in 1947, there were fewer than 300 large dams in India; but, now there are 5264 large dams completed and another 437 dams that are under construction (NRLD 2018) – besides several thousand smaller dams. India now ranks third in the world in dam building, after China and USA (ICOLD 2017).

Post-independence, a substantial number of dams have been added and close to 80% of India's large dams have now become more than 25 years old. Besides, there are 213 large dams which are over 100 years old. Figure-1-1 shows the journey of dam building activity in India.

A substantial proportion of Indian dams have now become old. As the dams age they deteriorate, thus posing a potential threat to life, health, property, and the environment. Lack of maintenance, upstream and downstream development etc. further amplify the problems. Many of these ageing dams have various structural deficiencies and shortcomings and they do not meet the requirements of the present design standards – both structurally and hydrologically. Thus, an increasing number of dams require rehabilitation. Safety of these dams is very important for safeguarding the national investments and the benefits derived.

Without proper maintenance, repairs, and rehabilitation, dams will not be able to provide their intended benefits and could be at risk for failure. State Govt. and other dam owning agencies need to identify deficiencies in dams through regular inspection programs, though inspections alone will not address safety concerns posed by inadequately maintained or deficient dams.

Some of the commonly encountered problems in Indian dams as also observed in the ongoing DRIP project are as under:

- Concrete/Masonry dams in poor condition with deficiencies such as cracking in dam, excessive seepage through dam body, improper foundation considerations, choking of drainage holes, poorly constructed lift joints, honey combing etc.
- Embankment dams in poor condition with deficiencies such as cracking in dam, piping, excessive seepage, disturbed rip-rap, rain cuts, vegetation etc.
- Structurally unsafe dams requiring structural strengthening;
- Damages on spillway crest, piers, training/divide walls, energy dissipaters (stilling basins, buckets etc.), downstream spill channel including

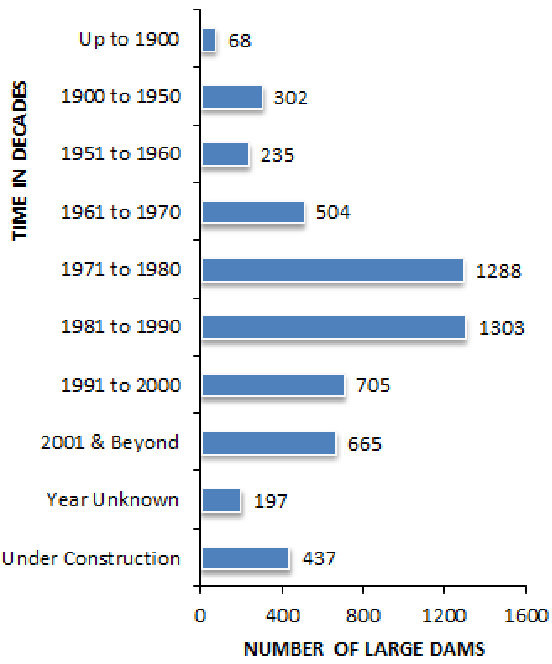

Figure 1-1: Distribution of large dams in India decade-wise (CWC 2018).

various miscellaneous structures etc.;

- Need for repairs and regular maintenance of the gates, hoists and other hydro-mechanical equipment's;
- Inadequate spillway capacity as seen through hydrological assessments, requiring structural/ non-structural measures;
- Poor condition of approach roads; communication facilities, dam instruments not in working condition, non-existence of emergency and disaster management plans.

For finalizing the rehabilitation measures various studies are required to be carried out e.g. hydrological review studies, reservoir routing studies, checking of the adequacy of existing freeboard, examination of various structural and non-structural alternatives in case the spillway capacity is found inadequate, investigations required for additional spillways construction including geological investigations, investigations required for issues like excessive seepage in masonry dams (Geo-physical examination to determine the seepage paths, low density areas etc.), Scanning of the upstream face of the dam for identification of cracks in Concrete dams (as required), material testing to determine various engineering parameters of existing dams/foundations, examination of instrumentation data, design studies including hydraulic designs, review of dam stability for different conditions, finalization of methodology and repair materials for undertaking repairs to damages etc.

1.1 Institutional Support for proper maintenance and rehabilitation of Dams

For ensuring continued benefits from dams, periodic surveillance and corrective measures based on sound design studies will be needed to be implemented by the States for which fund constraints are routinely experienced, as specific rehabilitation measures may far exceed the allocations available out of normal O&M funds. Adequate funds for maintenance and up-gradation of dams will be needed on regular basis to maintain these structures in good and safe condition.

There is an urgent need in the country for institutional strengthening in the field of dam safety and giving adequate importance to support design review & regular maintenance. Sensitizing the organization with the responsibility of operation and maintenance for the dam infrastructure is certainly more effective than to provide expensive aid later to undertake large rehabilitation works. Good quality maintenance works executed as per technical specifications are important in combating the need for expensive rehabilitation.

1.2 Purpose of this manual

The purpose of this Manual is to provide an overview of the current design practices and various checks and review studies necessary before undertaking dam rehabilitation works and to give enough references for the benefit of Indian Engineers.

This Manual which has been prepared under the Dam Rehabilitation and Improvement Project (DRIP) is to be used along with various other Manuals and Guidelines prepared/under preparation for promoting and ensuring Dam Safety for the dams in India.

1.3 Publication and Contact Information

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

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1.4 Acknowledgments

In preparing this manual, work of others in India, the United States, and elsewhere has been drawn from liberally. Grateful appreciation is extended to the following organizations whose publications and websites have given valuable information on various aspects of dam rehabilitation:

- International Commission on Large Dams (ICOLD)
- U.S. Army Corps of Engineers (USACE)
- U.S. Bureau of Reclamation (USBR)
- American Society of Civil Engineers (ASCE)
- Institution of Civil Engineers (ICE)
- Bureau of Indian Standards.
- Central Board of Irrigation & Power.

Further the Sections 6.10, 6.11, 6.12 and 6.13 as well as Appendix C of this Manual are based on the book:

Anil K Chopra, Earthquake Engineering for Concrete Dams: Analysis, Design and Evaluation, Wiley, 2019.

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Chapter 2. Hydrological Review And Flood Routing Studies

Dam Safety guidelines in India provide for a comprehensive dam safety evaluation of each dam by an independent panel of experts known as Dam Safety Review Panel (DSRP) once in 10 years or on occurrence of any extreme hydrological or seismic event or any unusual condition in the dam or in the reservoir rim.

Embankment dams are highly susceptible to failure if overtopped. For concrete dams the key hydrologic aspect is to know the level of overtopping, the duration of overtopping, and the return period for these events, because the loading can cause instability of the concrete dam. Increasing hydrostatic levels cause higher loads and stresses on the dam and foundation that can lead to sliding or overturning. Overtopping flows can erode the foundation causing instability.

The terms of reference of the comprehensive dam safety evaluation shall include but not be limited to;

1. General assessment of hydrologic and hydraulic conditions, review of design flood, flood routing for revised design flood and mitigation measures.
2. Review and analysis of available data of dam design including seismic safety, construction, operation, maintenance and performance of dam structure and appurtenant works.
3. Evaluation of procedures for operation, maintenance and inspection of dam and to suggest improvements / modifications.
4. Evaluation of any possible hazardous threat to the dam structure such as dam abutment slope stability failure or slope failures along the reservoir periphery

In view of the above the design flood for each dam in India is needed to be reviewed once in every 10 years approximately based on additional data.

The guidelines for carrying out design flood review study are being prepared separately.

2.1 Flood Routing Studies

After the design flood is reviewed and finalized, flood routing studies are required to be carried out to determine the Maximum Water Level and the routed Outflow in case the revised design flood is higher than the original design flood.

The adequacy of freeboard available above the revised Maximum Water Level is then needed to be checked. This aspect will be covered in the next chapter.

2.2 Basic Equation used

The basic equation used for routing the flood through the reservoir created by a dam is given below:

$$\left(\frac{I_1+I_2}{2}\right)t - \left(\frac{O_1+O_2}{2}\right)t = S_2 - S_1 = \Delta S$$

Where,

t = Time interval

I_1 = Inflow at the beginning of the time interval t.

I_2 = Inflow at the end of time interval t.

O_1 = Outflow at the beginning of the time interval t.

O_2 = Outflow at the end of time interval t.

S_1 = Gross storage at the beginning of the time interval t.

S_2 = Gross storage at the end of time interval t.

ΔS = Incremental Storage in time interval t.

2.3 Input Data Required

The data required for carrying out flood routing studies is given below:

i) Approved Inflow Design Flood Hydrograph

ii) Reservoir Storage vs Reservoir Elevation curve

iii) Reservoir Elevation vs Outflow curve

The inflow design flood hydrograph for use in flood routing is required to be taken from the design flood review study carried out for an existing dam.

For Reservoir Storage vs. Reservoir Elevation curve it may be desirable to carry out a fresh bathymetric survey of the existing dam, in case such a survey has not been carried out in the near future (say in the last 10 years) to account for the effects of reservoir sedimentation. The periodicity of the survey could however be based on site specific conditions. However in absence of the above, the original/earlier curve may have to be used for the study. Where necessary the curve may need to be suitably extrapolated.

Spillway outflow curve normally depends upon the type of spillway provided and on the hydraulics of flow over the control structure.

In an un-gated spillway (here the spillway crest level will be the FRL) there is no control over the flow. Here the flow varies with the head over the crest. Attenuation of flow occurs as the surcharge storage (Storage between FRL and MWL) increases with an increase in spillway discharge/reservoir level.

In a gated spillway the outflow can be varied with respect to reservoir head by operation of the gates up to FRL. One assumption for an operating gate controlled spillway may be that the gates are so regulated that the inflow is equal to outflow until the gates are wide open. Another assumption may be to open the gates at a slower rate so that the storage will accumulate before the gates are wide open. Depending on the requirements and purpose of the dam, operating rules are formulated which are contained in the Operating and Maintenance Manuals for dams.

It will be preferable to use the spillway outflow curve developed through hydraulic model studies at the time of initial designs while carrying out the flood routing studies. However where such data is not available it will need to be prepared afresh. For this purpose the design head used earlier to develop the spillway crest profile is required to be determined. This can be obtained from the equation of the downstream quadrant of the ogee spillway crest (Refer IS 6934 – Recommendations for hydraulic design of high ogee overfall spillways).In case this equation is also not available either fresh hydraulic model studies can be carried out to develop the spillway outflow curve or the design head may have to be suitably assumed say equal to Full Reservoir level minus the Spillway Crest level. The spillway outflows for different reservoir elevations may be determined as per various BIS codes/or as per any standard books on Hydraulics.

Outflows need not necessarily be limited to discharges through the spillway, but may be supplemented by releases through the outlets, under sluices etc. on a case to case basis depending upon their discharging capacity. In all such cases the size, type and method of operation of the spillway and outlets with reference to the storages or to the inflow must be pre-determined to establish an outflow – reservoir elevation relationship.

2.4 Commonly used methods for carrying out flood routing studies

The following two methods are commonly used:

i) Trial and Error Method

ii) Modified Puls Method

The Modified Puls method is generally preferred as it does not involve any trial and error procedure.

For gated spillways, normally the flood routing studies are carried out assuming the flood to impinge at FRL and assuming inflow to be equal to outflow at FRL.

For un-gated spillways the routing has to start from spillway crest elevation (FRL) and with inflow at time equal to 0.0 hours taken from the design inflow hydrograph.

Flood Routing studies are required to be carried out with all gates operative and also with 10% of the gates subject to a minimum of one gate as in-operative.

The maximum water level (MWL) corresponds to the condition when all the gates are taken as operative. (Refer IS 11223 – Guidelines for fixing spillway capacity).

2.4.1 Trial and Error Method

An example of flood routing study carried out by this method is illustrated at Table 2-1. The basic equation used is at para 2.2.

The data used which consists of Inflow Hydrograph, Reservoir Storage vs. Reservoir Elevation curve and the Spillway Discharge curve (Outflow curve) is at Figures 2-1, 2-2 and 2-3 respectively. An un-gated spillway was selected in the example illustrated.

The procedure for computations shown in Table 2-1 consists of the following steps:

i) Column (1) denotes the time in hours.

ii) Select a time interval and enter it in column (2).

iii) Obtain column (3) which is the inflow rate corresponding to the time in hours at column (1) from the inflow hydrograph.

iv) Column (4) represents average inflow for time interval.

v) Obtain column (5) i.e. inflow in volume units by multiplying average inflow rate obtained in column (4) with the time interval.

vi) Assume a trial reservoir water elevation in column (6). Determine the corresponding rate of outflow from the spillway outflow curve and record it in column (7).

vii) Average the rates of outflow determined at the beginning and at the end of the time interval, and enter this average value in column (8).

viii) Obtain column (9) i.e. outflow in volume units by multiplying average outflow rate obtained in column (8) with the time interval.

ix) Calculate change in storage in column (10) which is equal to column (5) – column (9).

x) The initial value in column (11) represents the reservoir storage at the beginning of the first time period. Determine value of reservoir storage at the end of time interval and enter it in column (11) by adding incremental storage obtained in column (10) to the previous value of column (11) which is the reservoir storage at the beginning of the time interval.

xi) Determine reservoir elevation in column (12) corresponding to storage in column (11).

xii) Compare reservoir elevation in column (12) with trial reservoir elevation assumed in column (6). If they do not agree, assume another trial reservoir elevation and repeat the procedure until such agreement is reached.

The trials have not been shown in Table 2-1. Only the final values are indicated.

xiii) The above steps are to be repeated till the end of inflow hydrograph.

In this way the outflow hydrograph, its peak value and the maximum reservoir elevation attained can be determined.

The results as can be seen from Table 2-1 are as under:

Peak inflow = 1030.00 cumec

Peak outflow = 359.50 cumec

Maximum water level attained = 322.67 m

2.4.2 Modified Puls Method

The basic equation used for routing the flood through the reservoir created by a dam is given below:

$$\left(\frac{I_1+I_2}{2}\right)t - \left(\frac{O_1+O_2}{2}\right)t = S_2 - S_1 = \Delta S$$

The symbols used in the above equation have been defined earlier in para 2.2.

This equation can be re written as under:

$$\left(\frac{I_1+I_2}{2}\right) + \left(\frac{S_1}{t} - \frac{O_1}{2}\right) = \left(\frac{S_2}{t} + \frac{O_2}{2}\right)$$

Modified Puls method in addition to the input data specified in para 2.3 envisages preparation of an additional curve between Outflow (O) and (S/t + O/2).

A gated spillway has been selected to illustrate this method.

The data used which consists of Inflow Hydrograph, Reservoir Storage vs. Reservoir Elevation curve, Spillway Discharge curve (Outflow curve) and Outflow (O) vs. (S/t + O/2) curve is at Figures 2-4, 2-5, 2-6 and 2-7 respectively.

The flood routing study is illustrated at Table 2-2.

First of all a curve between Outflow (O) and (S/t + O/2) is required to be prepared.

This is shown in Figure 2-7 (a) and 2-7 (b).

It envisages the following steps:

i) In column (2) enter the reservoir elevation.

ii) In column (3) the storage against the reservoir elevation considered in column (2) is recorded.

iii) A suitable time interval (t) is assumed.

iv) S/t in cumec is calculated in column (4) for the storage determined in column (3).

v) In column (5) the outflow (O) corresponding to the reservoir elevation assumed in column (2) is recorded.

vi) In column (6) the factor O/2 is recorded where O corresponds to the outflow determined in column (5).

vii) In column (7) the factor (S/t + O/2) is determined considering S/t and O/2 calculated in columns (4) and (6) respectively.

viii) This process is normally carried out for reservoir elevations from spillway crest level up to likely MWL for ungated spillways and from FRL to likely MWL for gated spillways.

After preparation of the above curve flood routing by Modified Puls method can be done.

As this is an illustration of a gated spillway flood routing studies are carried out assuming the flood to impinge at FRL and assuming inflow to be equal to outflow at FRL.

The steps involved are as under:

i) In the first row which corresponds to Sl. No. 1 write the FRL under Reservoir Elevation in column (8) in view of the assumption stated above.

ii) Write the outflow corresponding to the reservoir elevation (FRL in this case) in column (7).

iii) Corresponding to this outflow, determine the value of (S/t + O/2) from the curve between O and (S/t + O/2). Write this value in column (6).

iv) Under column (3) write the inflow value which will be equal to the outflow determined at step (ii) above.

v) Under column (2) write the value of time (in hrs.) which will be the time corresponding to the above inflow value as determined from the inflow hydrograph.

vi) This completes the first row of calculations at Sl. No.1.

vii) The same time interval selected while preparing the curve between O and (S/t + O/2) has to be used for further flood routing calculations.

viii) In the second row which corresponds to Sl. No. 2, the time in column (2) will be equal to the time determined under step (v) above plus the time interval discussed in the earlier step.

ix) For the time determined above find out the inflow rate from the inflow hydrograph and enter it in column (3) in the second row. This is the inflow at the end of the time interval.

x) Find the mean inflow which is the mean of the inflow rates at the beginning and at the end of the time interval i.e. the mean of the inflow values under column (3) at Sl. No. 1 and 2 in this case. Record it under column (4) at Sl.No.2.

xi) Find (S/t - O/2) at the beginning of the time interval by subtracting O from (S/t + O/2) determined in the row corresponding to Sl.no. 1 which corresponds to the values at the beginning of the time interval. Record it under column (5) in the row corresponding to Sl. No.2.

xii) Determine (S/t + O/2) at the end of the time interval by adding the mean inflow in column (4) and the (S/t - O/2) value at the beginning of the time interval in column (5) in Sl.no.2. Record it under column (6) corresponding to Sl. No.2.

xiii) Corresponding to the above (S/t + O/2) value at the end of the time interval find the value of Outflow at the end of the time interval from the curve between O and (S/t + O/2). Record it under column (7) corresponding to Sl. No.2.

iv) Corresponding to the above outflow value determine Reservoir Elevation from the Reservoir Elevation vs Outflow curve. Record it under column (8) corresponding to Sl. No.2.

v) The above steps are to be repeated till the end of inflow hydrograph.

In this way the outflow hydrograph, its peak value and the maximum reservoir elevation attained can be determined.

The results as can be seen from Table 2-2 are as under:

Peak inflow = 9832.29 cumec

Peak outflow = 4885.78 cumec

Maximum water level attained = 282.20 m

The inflow & outflow hydrographs are plotted in Figure 2-8.

Time (hr)	Inflow (Cumec)
0	100
1	1030
2	328
3	107
4	53
5	33
6	27
7	20
8	17
9	11
10	1

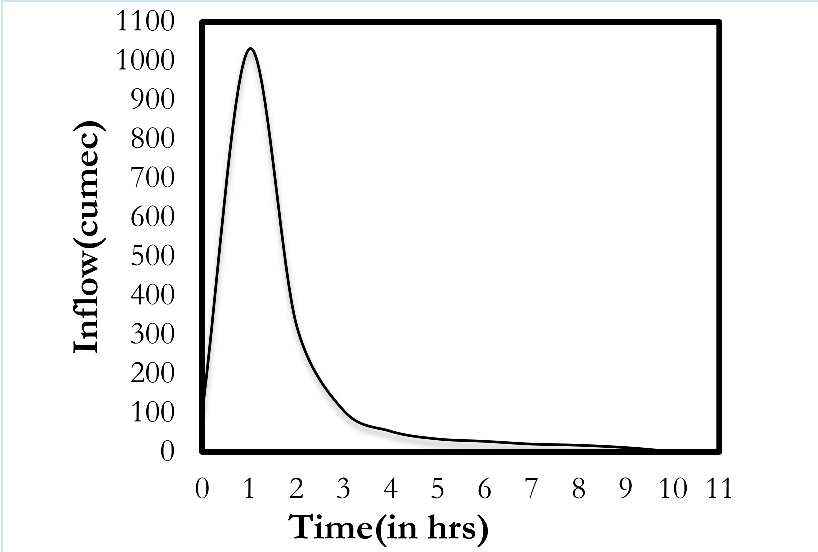


Figure 2-1: Inflow Hydrograph (Used in the example with Trial and Error Method)

Reservoir Elevation (m)	Reservoir Capacity (Mm^3)
318.20	0.00
319.23	2.00
321.85	14.73
323.07	19.89
324.90	29.18
325.40	31.72
325.90	34.26
326.40	36.80
326.90	39.34
327.40	41.87
327.90	44.41
328.40	46.95
328.90	49.49
329.40	52.03
329.90	54.56
330.40	57.10
330.90	59.64
331.40	62.18
331.90	64.72
332.40	67.26
332.90	69.79
333.40	72.33

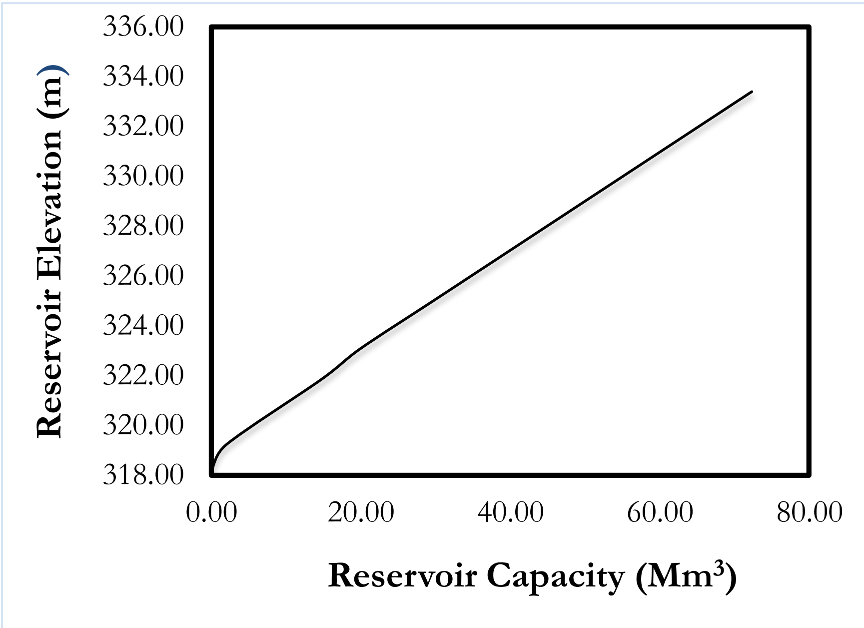


Figure 2-2: Reservoir storage vs Reservoir elevation Curve (Used in the example with Trial and Error Method)

Reservoir Elevation (m)	Spillway Discharge (cumec)
321.85	0.00
322.27	111.00
322.80	439.00
323.30	751.98
323.80	1064.95
324.30	1377.93
324.80	1690.91
325.30	2003.89
325.80	2316.86

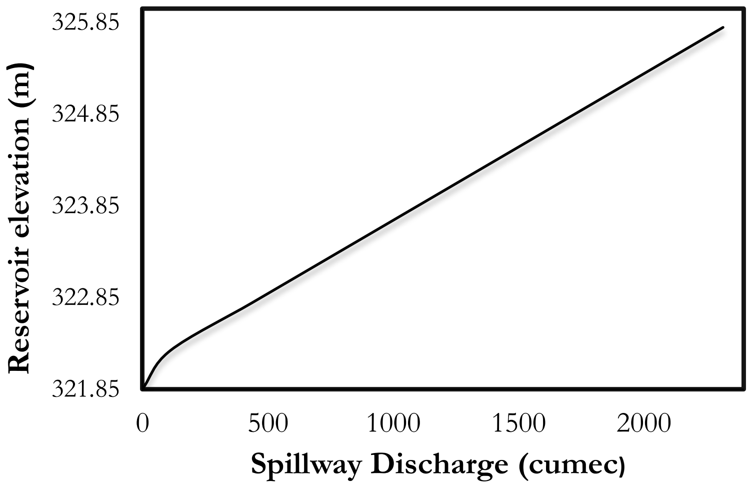


Figure 2-3: Outflow Curve (Used in the example with Trial and Error Method)

Table 2-1: Trial and Error Method for Flood Routing

Time (hr)	Time Interval t (hr)	Inflow (cumec)	Average rate of Inflow (cumec)	Inflow Volume During Time Interval (Mm^3)	Trial Reservoir Level (m)	Outflow (cumec)	Average Rate of Outflow (cumec)	Outflow Volume During Time Interval (Mm^3)	Change in Storage, ΔS During Time Interval (Mm^3)	Total Storage at the End of Time Interval (Mm^3)	Reservoir Elevation (m)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
0		100				0				14.73	321.85
1	1	1030	565.00	2.03	322.29	121.64	60.82	0.22	1.82	16.55	322.29
2	1	328	679.00	2.44	322.67	359.50	240.57	0.87	1.58	18.12	322.67
3	1	107	217.50	0.78	322.59	309.43	334.47	1.20	-0.42	17.70	322.59
4	1	53	80.00	0.29	322.45	221.79	265.61	0.96	-0.67	17.03	322.45
5	1	33	43.00	0.15	322.31	134.16	177.98	0.64	-0.49	16.55	322.31
6	1	27	30.00	0.11	322.27	110.21	122.19	0.44	-0.33	16.22	322.27
7	1	20	23.50	0.08	322.18	86.54	98.38	0.35	-0.27	15.95	322.18
8	1	17	18.50	0.07	322.16	81.28	83.91	0.30	-0.24	15.71	322.16

Results

Maximum water level attained	=	**322.67 m**
Peak rate of inflow	=	**1030 cumec**
Peak rate of outflow	=	**359.5 cumec**

Time (hr)	Inflow (cumec)	Time (hr)	Inflow (cumec)	Time (hr)	Inflow (cumec)
0	210.00	21	4862.26	42	3378.42
1	213.96	22	5377.58	43	2932.52
2	218.65	23	5854.68	44	2552.99
3	228.45	24	6325.47	45	2184.38
4	243.08	25	6775.38	46	1859.50
5	285.50	26	7300.67	47	1550.59
6	339.18	27	7839.03	48	1298.00
7	415.67	28	8441.95	49	1076.01
8	497.85	29	9031.16	50	894.15
9	612.77	30	9557.69	51	701.39
10	737.46	31	9820.25	52	549.95
11	901.63	32	9832.29	53	432.37
12	1082.55	33	9514.37	54	326.10
13	1288.83	34	8994.15	55	268.96
14	1539.62	35	8229.41	56	237.46
15	1844.18	36	7422.15	57	223.35
16	2211.47	37	6581.64	58	214.73
17	2683.09	38	5812.36	59	210.00
18	3227.54	39	5095.01	60	210.00
19	3778.52	40	4474.41	61	210.00
20	4321.68	41	3898.98		

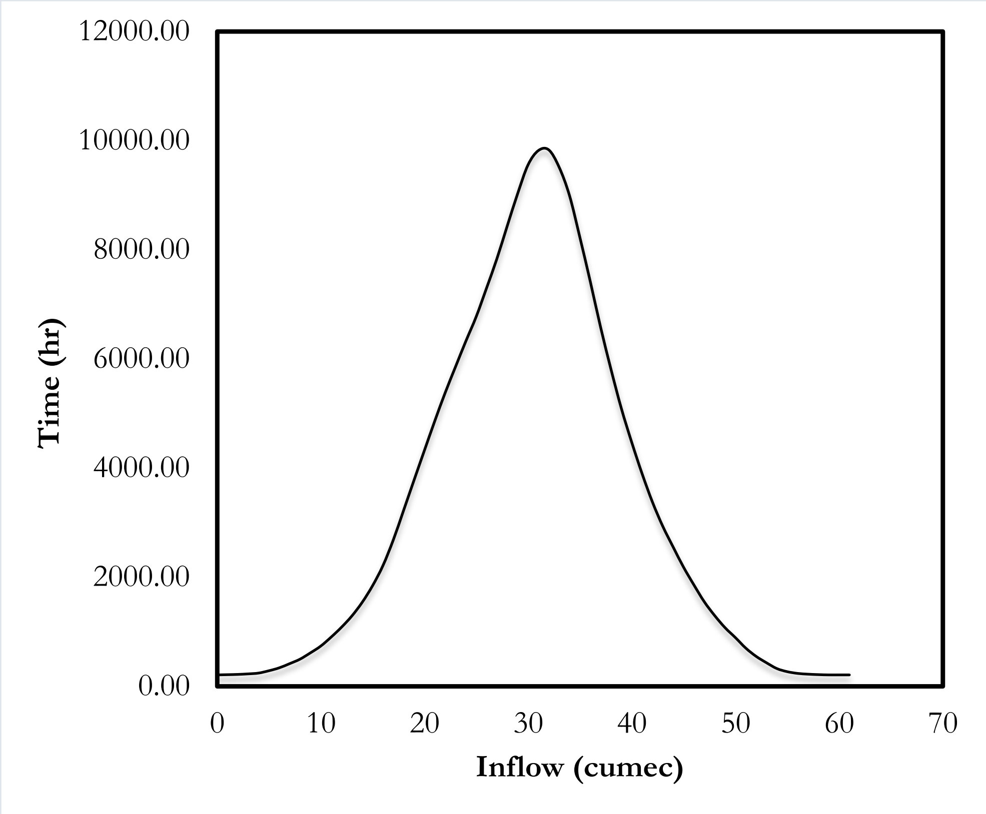


Figure 2-4: Inflow Hydrograph (Used in the example with Modified Puls Method)

Sl. No.	Elevation (m)	Capacity (Mm³)	Sl. No.	Elevation (m)	Capacity (Mm³)	Sl. No.	Elevation (m)	Capacity (Mm³)
1	274.32	523.42	22	278.52	786.739	43	282.61	1100.423
2	274.52	534.509	23	278.72	801.075	44	282.81	1116.026
3	274.72	545.702	24	278.89	813.49	45	282.99	1130.0687
4	274.92	557.007	25	279.09	827.963	46	283.49	1169.0762
5	275.12	568.519	26	279.29	842.711	47	283.99	1208.0837
6	275.32	580.134	27	279.49	857.606	48	284.49	1247.0912
7	275.52	591.873	28	279.69	872.73	49	284.99	1286.0987
8	275.72	603.82	29	279.89	887.966	50	285.49	1325.1062
9	275.92	615.873	30	280.09	903.34	51	282.61	1100.423
10	276.12	628.064	31	280.29	918.943	52	282.81	1116.026
11	276.32	640.453	32	280.41	928.79	53	282.99	1130.0687
12	276.52	652.95	33	280.61	944.393	54	283.49	1169.0762
13	276.72	665.601	34	280.81	959.996			
14	276.92	678.432	35	281.01	975.599			
15	277.12	691.381	36	281.21	991.202			
16	277.32	704.517	37	281.41	1006.805			
17	277.52	717.828	38	281.61	1022.408			
18	277.72	731.255	39	281.81	1038.011			
19	277.92	744.877	40	282.01	1053.614			
20	278.12	758.676	41	282.21	1069.217			
21	278.32	772.598	42	282.41	1084.82			

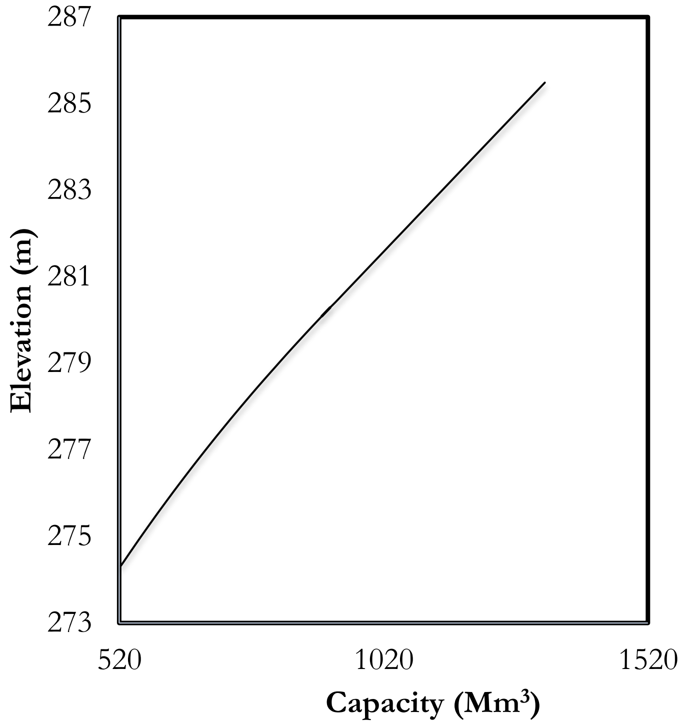


Figure 2-5: Reservoir Elevation - Capacity Curve (Used in the example with Modified Puls Method)

Sl. No.	Reservoir Elevation (m)	Spillway Discharge (cumec)	Sl. No.	Reservoir Elevation (m)	Spillway Discharge (cumec)	Sl. No.	Reservoir Elevation (m)	Spillway Discharge (cumec)
1	274.32	0.00	21	278.00	1425.70	41	282.00	4681.73
2	274.50	2.73	22	278.20	1551.15	42	282.20	4884.67
3	274.70	15.45	23	278.40	1680.93	43	282.40	5091.39
4	274.90	42.54	24	278.60	1815.00	44	282.60	5301.89
5	275.00	62.52	25	278.80	1953.33	45	282.80	5516.16
6	275.10	87.27	26	279.00	2095.91	46	282.90	5617.64
7	275.20	117.11	27	279.20	2242.52	47	283.00	5730.43
8	275.40	193.27	28	279.40	2391.49	48	283.10	5837.56
9	275.60	276.03	29	279.60	2544.42	49	283.20	5944.70
10	275.80	344.46	30	279.80	2701.28	50	283.30	6051.83
11	276.00	418.43	31	280.00	2862.05	51	283.40	6158.97
12	276.20	497.74	32	280.20	3026.71	52	283.50	6266.10
13	276.40	582.24	33	280.40	3195.25	53	283.60	6373.24
14	276.60	671.79	34	280.60	3367.66	54	283.70	6480.37
15	276.80	766.10	35	280.80	3543.91	55	283.80	6587.51
16	277.00	864.68	36	281.00	3724.01	56	283.90	6694.64
17	277.20	967.88	37	281.20	3907.93	57	284.00	6801.78
18	277.40	1075.65	38	281.40	4095.67	58	284.10	6908.91
19	277.60	1187.90	39	281.60	4287.23	59	284.20	7016.04
20	277.80	1304.60	40	281.80	4482.58			

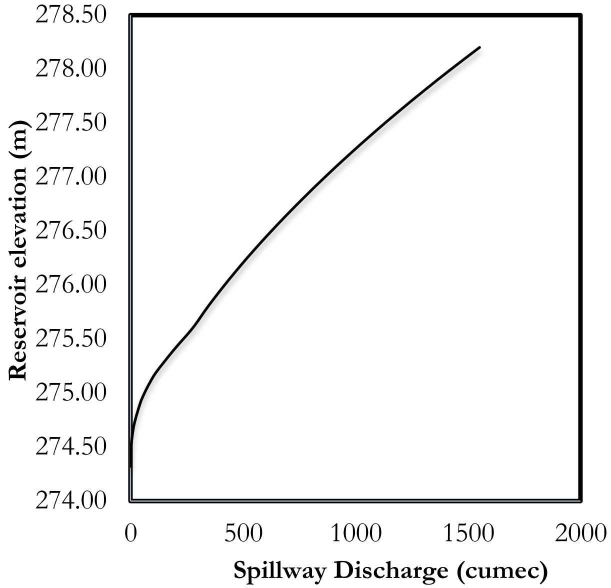


Figure 2-6: Outflow Curve (Used in the example with Modified Puls Method)

Sl. No.	Reservoir Level (m)	Storage(S) (Mm³)	S/t (cu-mec)	Outflow (O) (cumec)	O/2 (cumec)	S/t +O/2 (cumec)	Sl. No.	Reservoir Level (m)	Storage(S) (Mm³)	S/t (cu-mec)	Outflow (O) (cu-mec)	O/2 (cu-mec)	S/t +O/2 (cumec)
Col.1	Col.2	Col.3	Col.4	Col.5	Col.6	Col.7	Col.1	Col.2	Col.3	Col.4	Col.5	Col.6	Col.7
1	274.32	523.42	145394.44	0.00	0.00	145394.44	31	280.00	896.42	249006.03	2862.05	1431.02	250437.05
2	274.50	533.40	148166.69	2.73	1.37	148168.06	32	280.20	911.92	253311.57	3026.71	1513.36	254824.93
3	274.70	544.58	151272.97	15.45	7.73	151280.70	33	280.40	927.97	257769.28	3195.25	1597.63	259366.91
4	274.90	555.88	154410.14	42.54	21.27	154431.41	34	280.60	943.61	262114.68	3367.66	1683.83	263798.51
5	275.00	561.61	156003.28	62.52	31.26	156034.54	35	280.80	959.22	266448.85	3543.91	1771.96	268220.80
6	275.10	567.37	157602.17	87.27	43.64	157645.80	36	281.00	974.82	270783.01	3724.01	1862.00	272645.02
7	275.20	573.17	159212.50	117.11	58.56	159271.06	37	281.20	990.42	275117.18	3907.93	1953.97	277071.15
8	275.40	584.83	162452.67	193.27	96.63	162549.30	38	281.40	1006.02	279451.35	4095.67	2047.84	281499.18
9	275.60	596.65	165736.61	276.03	138.01	165874.62	39	281.60	1021.63	283785.51	4287.23	2143.61	285929.13
10	275.80	608.64	169067.00	344.46	172.23	169239.23	40	281.80	1037.23	288119.68	4482.58	2241.29	290360.97
11	276.00	620.75	172430.39	418.43	209.22	172639.61	41	282.00	1052.83	292453.85	4681.73	2340.87	294794.71
12	276.20	633.02	175838.78	497.74	248.87	176087.65	42	282.20	1068.44	296788.01	4884.67	2442.34	299230.35
13	276.40	645.45	179292.17	582.24	291.12	179583.28	43	282.40	1084.04	301122.18	5091.39	2545.70	303667.88
14	276.60	658.01	182780.67	671.79	335.90	183116.56	44	282.60	1099.64	305456.35	5301.89	2650.95	308107.29
15	276.80	670.73	186314.83	766.10	383.05	186697.88	45	282.80	1115.25	309790.51	5516.16	2758.08	312548.59
16	277.00	683.61	189892.11	864.68	432.34	190324.45	46	282.90	1123.05	311957.60	5617.64	2808.82	314766.42
17	277.20	696.64	193509.83	967.88	483.94	193993.78	47	283.00	1130.85	314124.68	5730.43	2865.21	316989.90
18	277.40	709.84	197178.17	1075.65	537.82	197715.99	48	283.10	1138.65	316291.76	5837.56	2918.78	319210.55
19	277.60	723.20	200888.56	1187.90	593.95	201482.51	49	283.20	1146.45	318458.85	5944.70	2972.35	321431.20
20	277.80	736.70	204639.94	1304.60	652.30	205292.24	50	283.30	1154.25	320625.93	6051.83	3025.92	323651.85
21	278.00	750.40	208443.50	1425.70	712.85	209156.35	51	283.40	1162.05	322793.01	6158.97	3079.48	325872.50
22	278.20	764.24	212290.22	1551.15	775.58	213065.80	52	283.50	1169.86	324960.10	6266.10	3133.05	328093.15
23	278.40	778.25	216181.78	1680.93	840.46	217022.24	53	283.60	1177.66	327127.18	6373.24	3186.62	330313.80
24	278.60	792.47	220131.50	1815.00	907.50	221039.00	54	283.70	1185.46	329294.26	6480.37	3240.19	332534.45
25	278.80	806.92	224143.71	1953.33	976.67	225120.37	55	283.80	1193.26	331461.35	6587.51	3293.75	334755.10
26	279.00	821.45	228180.60	2095.91	1047.95	229228.55	56	283.90	1201.06	333628.43	6694.64	3347.32	336975.75
27	279.20	836.07	232242.89	2242.52	1121.26	233364.15	57	284.00	1208.86	335795.51	6801.78	3400.89	339196.40
28	279.40	850.90	236362.01	2391.49	1195.75	237557.76	58	284.10	1216.67	337962.60	6908.91	3454.46	341417.05
29	279.60	865.92	240534.50	2544.42	1272.21	241806.71	59	284.20	1224.47	340129.68	7016.04	3508.02	343637.70
30	279.80	881.11	244752.72	2701.28	1350.64	246103.36							

Figure 2-7 (a) Storage Indication Table (Used in the example with Modified Puls Method)

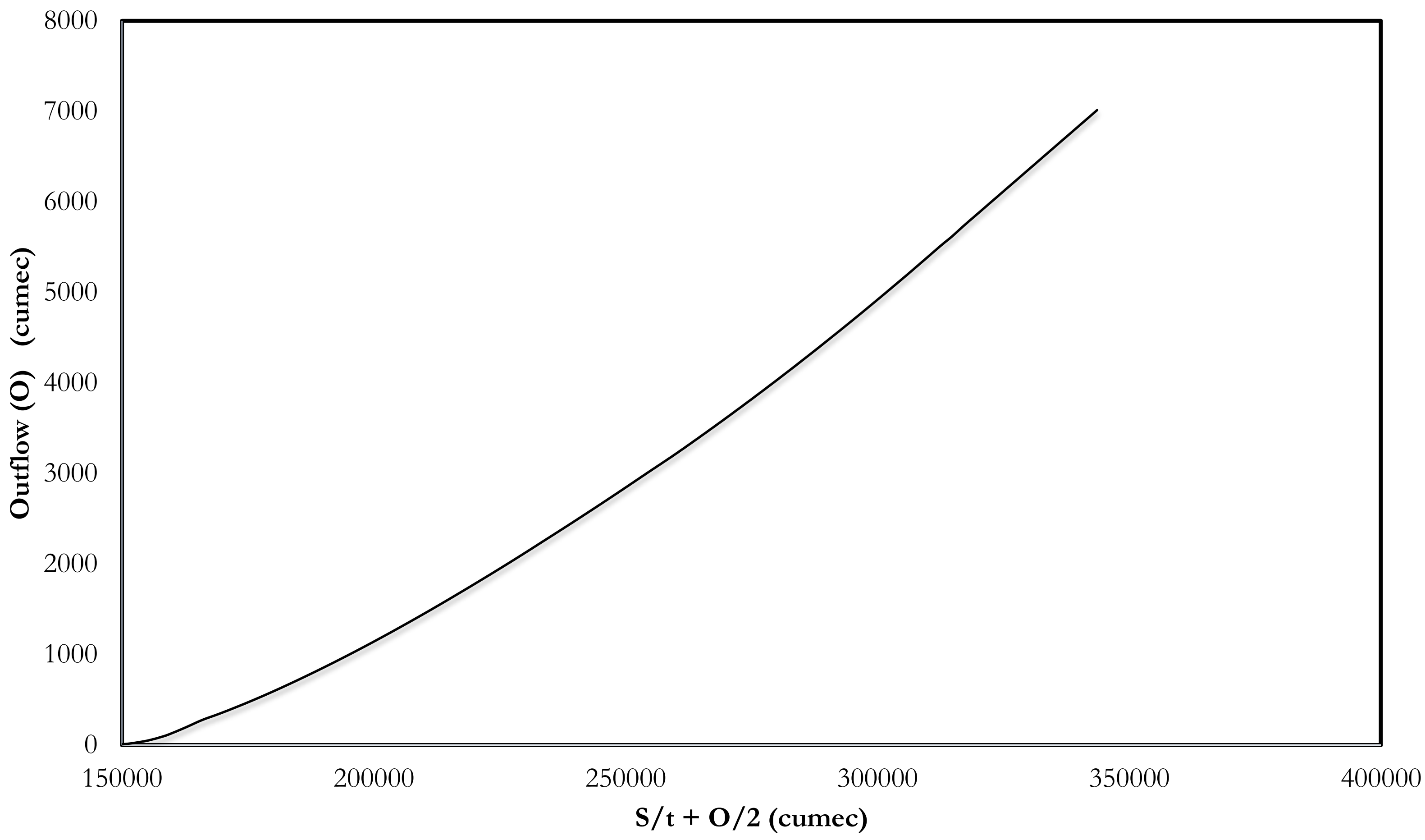
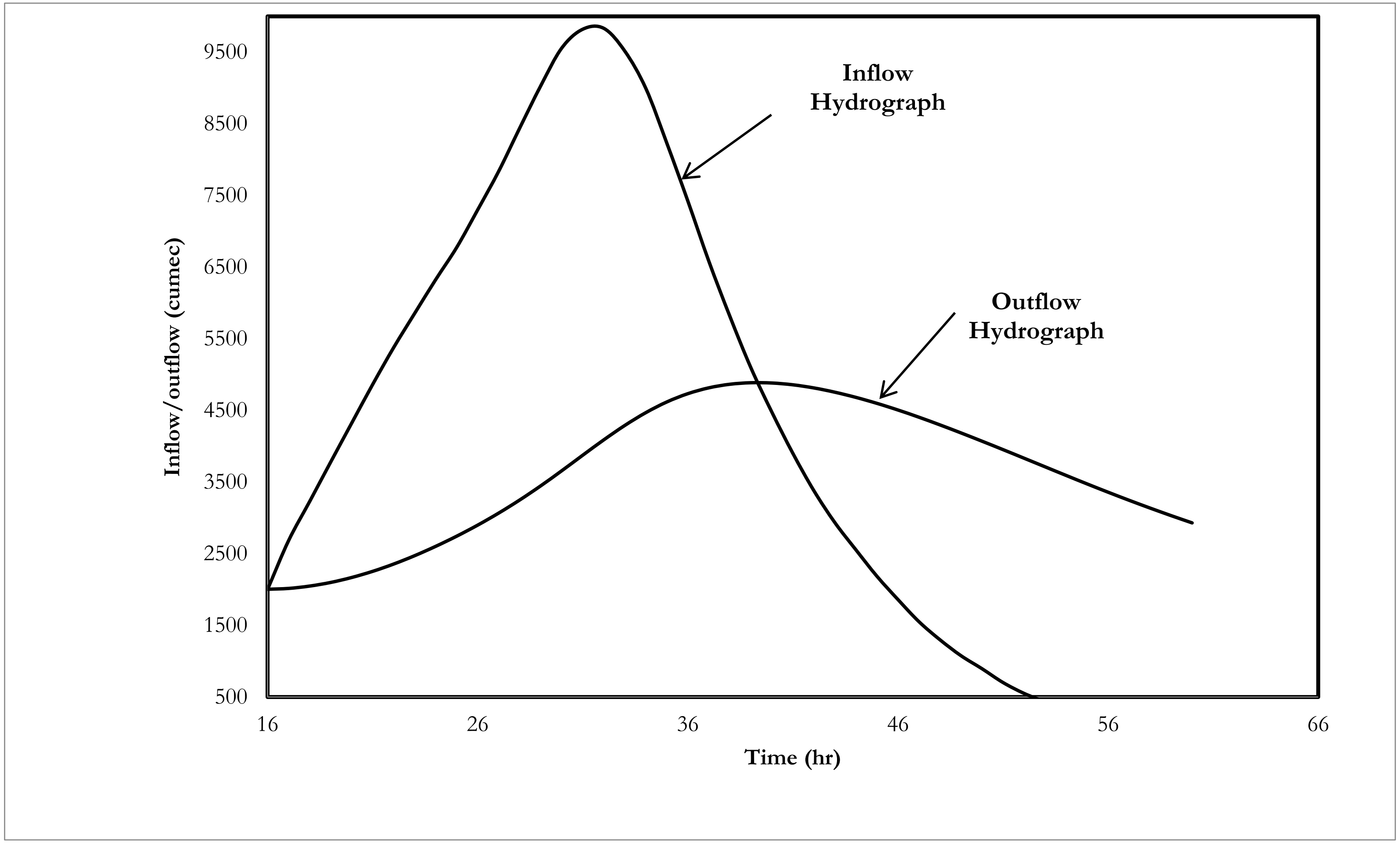


Figure 2-7 (b): Storage Indication Curve (Used in the example with Modified Puls Method)

Table 2-2: Reservoir Routing Table (Used in the example with Modified Puls Method)

Sl. No.	Time (hr)	Inflow (cumec)	Mean-flow (cumec)	S/t - O/2 (cumec)	S/t + O/2 (cumec)	Outflow (cumec)	Reser-voir Eleva-tion (m)	Sl. No.	Time (hr)	Inflow (cumec)	Mean-flow (cumec)	S/t - O/2 (cumec)	S/t + O/2 (cumec)	Outflow (cumec)	Reservoir Elevation (m)
Col.1	Col.2	Col.3	Col.4	Col.5	Col.6	Col.7	Col.8	Col.1	Col.2	Col.3	Col.4	Col.5	Col.6	Col.7	Col.8
1	16.0	2003.23			226558.24	2003.23	278.87	24	39.0	5095.01	5453.68	301405.02	306858.70	4885.78	282.20
2	17.0	2683.09	2343.16	224555.00	226898.16	2015.03	278.89	25	40.0	4474.41	4784.71	301972.93	306757.63	4882.14	282.20
3	18.0	3227.54	2955.32	224883.13	227838.45	2047.66	278.93	26	41.0	3898.98	4186.69	301875.49	306062.19	4857.11	282.17
4	19.0	3778.52	3503.03	225790.79	229293.82	2098.22	279.00	27	42.0	3378.42	3638.70	301205.08	304843.78	4813.26	282.13
5	20.0	4321.68	4050.10	227195.60	231245.70	2167.42	279.10	28	43.0	2932.52	3155.47	300030.52	303185.99	4753.59	282.07
6	21.0	4862.26	4591.97	229078.28	233670.25	2253.39	279.21	29	44.0	2552.99	2742.75	298432.40	301175.16	4681.22	282.00
7	22.0	5377.58	5119.92	231416.86	236536.78	2355.22	279.35	30	45.0	2184.38	2368.68	296493.94	298862.63	4597.98	281.92
8	23.0	5854.68	5616.13	234181.56	239797.69	2472.11	279.51	31	46.0	1859.50	2021.94	294264.64	296286.59	4505.27	281.82
9	24.0	6325.47	6090.07	237325.57	243415.65	2602.33	279.67	32	47.0	1550.59	1705.05	291781.32	293486.37	4404.48	281.72
10	25.0	6775.38	6550.42	240813.31	247363.74	2744.43	279.85	33	48.0	1298.00	1424.30	289081.89	290506.18	4297.22	281.61
11	26.0	7300.67	7038.03	244619.30	251657.33	2898.97	280.04	34	49.0	1076.01	1187.00	286208.97	287395.97	4185.27	281.49
12	27.0	7839.03	7569.85	248758.36	256328.21	3067.08	280.25	35	50.0	894.15	985.08	283210.70	284195.78	4070.09	281.37
13	28.0	8441.95	8140.49	253261.13	261401.62	3249.68	280.46	36	51.0	701.39	797.77	280125.69	280923.46	3952.31	281.25
14	29.0	9031.16	8736.56	258151.94	266888.49	3447.17	280.69	37	52.0	549.95	625.67	276971.14	277596.81	3832.58	281.12
15	30.0	9557.69	9294.43	263441.32	272735.75	3657.62	280.93	38	53.0	432.37	491.16	273764.23	274255.39	3712.32	280.99
16	31.0	9820.25	9688.97	269078.13	278767.10	3874.70	281.16	39	54.0	326.09	379.23	270543.08	270922.31	3592.35	280.85
17	32.0	9832.29	9826.27	274892.39	284718.66	4088.91	281.39	40	55.0	268.96	297.53	267329.95	267627.48	3473.77	280.72
18	33.0	9514.37	9673.33	280629.75	290303.08	4289.91	281.60	41	56.0	237.46	253.21	264153.72	264406.93	3357.85	280.59
19	34.0	8994.15	9254.26	286013.17	295267.43	4468.58	281.79	42	57.0	223.35	230.40	261049.08	261279.48	3245.29	280.46
20	35.0	8229.41	8611.78	290798.85	299410.63	4617.71	281.94	43	58.0	214.73	219.04	258034.20	258253.23	3136.37	280.33
21	36.0	7422.15	7825.78	294792.92	302618.70	4733.17	282.05	44	59.0	210.00	212.36	255116.86	255329.23	3031.13	280.21
22	37.0	6581.64	7001.89	297885.53	304887.42	4814.83	282.13	45	60.0	210.00	210.00	252298.10	252508.10	2929.59	280.08
23	38.0	5812.36	6197.00	300072.60	306269.59	4864.57	282.18								



Results

1.	Maximum water level attained (m)	:	282.20
2.	Peak rate of inflow (cumec)	:	9832.29
3.	Peak rate of outflow (cumec)	:	4885.78
4.	TBL (m)	:	285.06

Figure 2-8: Inflow & Outflow Hydrograph (Used in the example with Modified Puls Method)

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Chapter 3. FREEBOARD ASPECTS

While carrying out a dam safety review of an existing dam, after the design flood is finalized and Maximum Water Level (MWL) worked out by flood routing, the adequacy of freeboard available above the revised MWL is required to be checked.

Freeboard is the vertical distance between a specified reservoir water surface elevation and the top of the dam, without camber.

Normal freeboard is defined as the difference in elevation between the top of the dam without camber and the higher of the top of conservation storage or top of joint - use storage as established from design requirements, which in India is normally called Full Reservoir level (FRL) i.e. the top of the spillway gates elevation for gated spillways or the spillway crest elevation for un-gated spillways .

Minimum freeboard is defined as the difference in elevation between the top of the dam without camber and the Maximum reservoir water surface that would result from routing the IDF (inflow design flood) through the reservoir.

The objective of having freeboard is to provide needed assurance against overtopping resulting from:

i) Wind set up and wave run-up

ii) Land slide and seismic motion

iii) Settlement

iv) Malfunction of Hydro-mechanical equipment's (Gates/Hoists)

v) Other uncertainties in design, construction and operation.

3.1 Methods of Freeboard Calculation

Various methods have been used for calculating freeboard. A lot of research work has taken place worldwide from time to time. Some of the methods used to calculate freeboard in dams are as under:

i. Stevenson's Formula as modified by Molitor.
ii. Guidelines as per USBR – Design of Small Dams (1987).
iii. USBR - ACER Technical Memorandum no. 2 - Freeboard Criteria and Guidelines for computing Freeboard allowances for storage dams (1981).
iv. USBR - ACER Technical Memorandum no. 2 - Freeboard Criteria and Guidelines for computing Freeboard allowances for storage dams (1992)
v. USBR – Design Standards No.13 – Embankment Dams, Chapter 6: Freeboard, September 2012.
vi. IS-10635 - Free board requirements in Embankments dam and IS 6512 - Design of Solid Gravity dams.

3.1.1 Stevenson's Formula as Modified by Molitor

Formulas for wave heights proposed by Stevenson have been widely used earlier. Details are available in reference - Thomas Stevenson, Design and Construction of Harbours: A treatise on Maritime Engineering, Edition 2, Edinburgh, 1874.

Molitor in the year 1935 proposed modifications in the Stevenson's formulas to include the wind velocity as follows:

$$\boldsymbol{hw = 0.017\sqrt{VF} + 2.5 - \sqrt[4]{F}}$$

where,

h_w = Height of wave in feet

F = Fetch or straight length of water subject to wind action in statute miles

V = Wind Velocity in miles/hr.

For F greater than 20 miles, the above equation was simplified as under:

$h_w = 0.17\sqrt{\boldsymbol{VF}}$

Details are available in reference – D. A. Molitor, "Wave Pressures on Sea Wall and Break Waters", Transactions American Society of Civil Engineers, Volume 100, 1935.

For earth dams with flat slopes, it was being assumed that the wave will ride up a vertical distance above still water level equal to 1.4 h_w to 1.5 h_w.

However these formulas are purely empirical and were not prescribed to be used outside the range for which they were tested i.e. for moderately deep water and wind velocities in excess of 60 miles per hour (Refer: Engineering for dams - Creager, Justin and Hinds, Volume 3).

These formulae (converted to MKS units) were also included in IS 6512 - Design of Solid Gravity Dams – 1972 edition for calculating freeboard.

3.1.2 Guidelines as per USBR – Design of Small Dams (1987 edition)

The USBR – Design of Small Dams (1987 edition) contains guidelines for determining freeboard for small earth fill dams.

A summary of empirical formulas proposed in this method for determining wave heights is reported to have been taken from American Society of Civil Engineers report ("Review of Slope Protection Methods," Subcommittee on Slope Protection, Soil Mechanics and Foundation Division, Proceedings ASCE, Vol. 74, June 1948). Table 3-1 was extracted by USBR from this report.

For the design of small dams with riprapped slopes, it has been recommended that the freeboard should be sufficient to prevent overtopping of the dam from wave run-up equal to 1.5 times the height of the wave as interpolated from Table 3-1, measured vertically from the still water level. Normal freeboard has been based on a wind velocity of 100 mile/hr. and minimum freeboard on a velocity of 50 mile/hr. Based on these assumptions and other considerations, Table 3-2 lists the least amount recommended for both normal and minimum freeboard on riprapped earth fill dams. The design of the dam should satisfy the most critical requirement. An increase in the freeboard shown in Table 3-2 for dams where the fetch is 2.5 miles and less maybe required if the dam is located in a very cold or a very hot dry climate, particularly if CL and CH soils are used for construction of the cores. It has also been recommended that the amount of freeboard shown in Table 3-2 be increased by 50 percent if a smooth pavement is to be provided on the upstream slope.

The above methods for determining freeboard requirements are generally found adequate for small dams.

Table 3-1: Fetch and Wind velocity versus Wave Height

Fetch, mile	Wind Velocity, mile/hr	Wave Height, ft
1	50	2.7
1	75	3.0
2.5	50	3.2
2.5	75	3.6
2.5	100	3.9
5	50	3.7
5	75	4.3
5	100	4.8
10	50	4.5
10	75	5.4
10	100	6.1

Table 3-2: Fetch versus recommended Normal and Minimum freeboard

Fetch, mile	Normal freeboard, ft	Minimum freeboard, ft
<1	4	3
1	5	4
2.5	6	5
5	8	6
10	10	7

3.1.3 USBR - ACER Technical Memorandum no. 2 - Freeboard Criteria and Guidelines for computing Freeboard allowances for Storage dams (1981)

As per this memorandum the minimum freeboard should not be less than 3 feet for embankment dams i.e. between the top of the dam without camber and the Maximum reservoir water surface that would result from routing the IDF (inflow design flood) through the reservoir.

For concrete dams zero minimum freeboard is considered acceptable. Maximum reservoir water surface that would result from routing the IDF (inflow design flood) through the reservoir could extend up to the top of dam in most cases, where a standard 3.5 foot high solid parapet wall is constructed.

Use of parapet walls to provide freeboard allowances for earth dams can be allowed on a case-by-case basis. However the following safeguards must be met:

a. The maximum water surface resulting from routing the Inflow Design Flood (IDF) must not exceed the top of the impervious zone .

b. The parapet wall may only replace the portion of the freeboard needed to prevent overtopping from wave run-up**.**

c. Future foundation and embankment settlement that would adversely affect the structural integrity of the parapet wall should be allowed to occur prior to construction of the wall or the wall design should allow for future settlement .

The general freeboard criteria should be applied to existing as well as proposed dams taking into account conditions that have changed since the initial freeboard design determination. For example, settlement of the embankment and landslides, with the exception of that due to seismic shaking, would probably have occurred and may not need to be considered for existing dams. Addition of a parapet wall may be a feasible method of providing freeboard in some cases.

Additionally, the risk of malfunction of spillways and outlet works in existing dams is better known than at the time of original design because of maintenance and operating experience. When assessing the risk of malfunction, known limitations to gate operation as well as improvements in mechanical and electrical features or added provisions for attendance during periods of skilled operation should be considered. While 3 feet of freeboard has been established as the minimum criterion for proposed embankment dams, an evaluation of conditions at existing dams may indicate that some encroachment is acceptable.

A summary of the step-by-step procedure is given as under:

(i) Effective Fetch: In dam reservoir area the fetches at different locations/directions are limited by the land forms surrounding the body of water (reservoir contour). Shore lines are irregular and a general method was recommended to calculate the effective fetch.

The effective fetch at a given station can be computed as under:

$$Fe = \frac{\sum X_i cos\alpha_i}{\sum cos\ \alpha_i}$$

where,

α_i= Angle between the central radial and radial i

$X_i =$ Length of projection of radial i on the central radial.

A trial and error method was recommended in determining the effective fetch (maximum value to be considered in freeboard calculations). A few stations along the dam axis were needed to be considered for the purpose (see Figure 3-1).

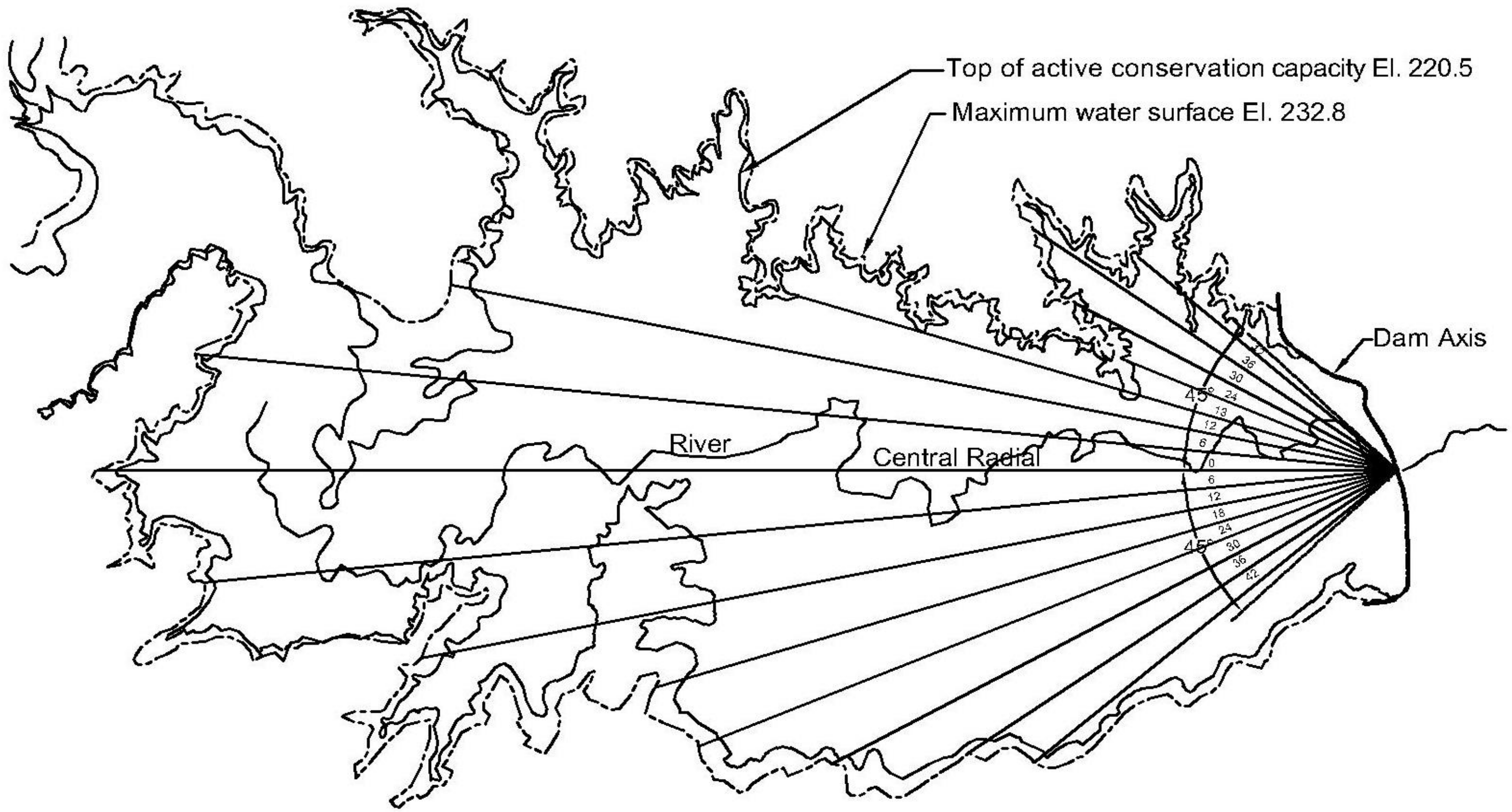


Figure 3-1: Effective Fetch Calculation

Radials up to 45° on either side of the central radial are to be taken (normally 7 radials at an angle of 6° each are taken thus covering an angular distance of 42° each on either side of the central radial).

(ii) Wind Velocity: The procedure utilizes the generalized fastest mile (approximate 1-minute duration) and 1-hour winds data (at 25 feet above ground level) of the United States based on the location of the reservoir if detailed project specific wind data are not available. Detailed site-specific wind data, if available, is preferred. The 2-hour wind velocity is estimated by multiplying the 1 hour wind velocity by a factor of 0.96. These overland velocities are then adjusted to over water velocities. An adjustment for over water winds can be carried out by multiplying the over land winds by the velocity ratios given below in Table 3-3.

After adjusting the overland velocities to over water velocities, a curve between wind velocity over water and wind duration is drawn. For this the fastest mile is taken as velocity with 1 min. duration, 1 hour velocity as of 1 hr. duration and 2 hour velocity as of 2 hr. duration.

(iii) Another curve between wind velocity over water and wind duration at a given reservoir can be developed based on figure 9 of this technical memorandum corresponding to the effective fetch computed in step (i).

(iv) The intersection of the wind velocity-duration curves developed in steps (ii) and (iii) above for the dam will determine the design wind velocity and its duration.

(v) The significant wave height can be estimated from figure 9 of this technical memorandum and the wave period from figure 10 of this technical memorandum based on design wind velocity determined in step (iv) and effective fetch determined in step (i).

(vi) The deep water wave length in feet can be computed by,

$$L=5.12\,T^{2}$$

where,

T= the wave period in seconds from step (v).

Most dams have relatively deep reservoirs compared to the wind generated wave length and the wave is unaffected by the reservoir floor. The equation given under this step is valid when the reservoir depth is deeper than one half of the wave length. If reservoir depth becomes a limiting factor, adjustment to L can be made by following procedure given in Shore Protection Manual, Volume – III, Coastal Engineering Research Center, U.S. Army Corps of Engineers, 1977.

(vii) Run-up from a significant wave on an embankment with rip-rap surface is given by:

$$R_S=\frac{H_S}{0.4+\left(\frac{H_S}{L}\right)^{0.5}\cot\theta}$$

where,

Hs = Significant wave height in feet.

Where the Significant wave height and the design wave height are not

Table 3-3: Wind relationship – Water to Land

Effective Fetch in miles	0.5	1	2	3	4	5 or more
Wind velocity Ratio (Over water)/(Over land)	1.08	1.13	1.21	1.26	1.28	1.30

equal, the design wave height is to be considered in the above equation.

L = Wave length in feet from step (vi)

$\theta =$ Angle of the upstream face of dam with horizontal.

This equation should not be used for slopes flatter than 1 (V): 5(H). For embankments dams with soil cement or other smooth upstream faces, the run-up computed by the above equation should be multiplied by a factor of up to 1.5, depending on the smoothness of the surface

Further the equation given in this step should not be used for computing run-up for rock fill dams Rock fill acts more like a rubble mound structure and has a different effect on energy dissipation than riprap placed on an impervious embankment. Run-up for rockfi1l dams may be determined from figure 11 of USBR technical memorandum.

For smooth impermeable slopes of concrete and other smooth surface dams with water depth at the dam (d_s) greater than three times the wave height (H_0), the relationship between wave run-up and wave height can be determined from figure 12 of USBR technical memorandum.

Results predicted by figure 12 of USBR technical memorandum are probably less than the run up on prototype structures because of scale effects due to the inability to scale roughness effects into small -scale laboratory tests. Run-up values from figure 12 of USBR technical memorandum should be adjusted for scale effects by using a factor obtained from figure 13 of USBR technical memorandum.

If the direction of wave propagation as defined by the central radial is not normal to the dam, a correction factor should be applied to the computed run-up. This factor consists of multiplying the computed run-up by the cosine of the angle between the wave propagation direction and a line normal to the dam as long as the angle is less than about 50°.

(viii) The wind setup in feet is given by:

$$S=\frac{U^2F}{1400\ D}$$

Where,

U = Design wind velocity over water in miles per hour from step (iv).

F = Maximum Wind fetch in miles

D = Average water depth along the central radial in feet.

(ix) The minimum freeboard requirement for wind-generated waves is the sum of wave run-up and wind setup and should be determined using moderate winds. These represent winds in terms of velocity, duration, direction, and seasonal distribution that may reasonably occur concurrently with maximum pool levels. If the response time between the design storm and the resulting maximum pool elevation is short, high winds that are sometimes associated with storms which may not have subsided must be considered in determining freeboard requirements. If the response time is longer than the storm period, a lower, more moderate wind would be appropriate.

If adequate data for probability analyses are available, a moderate wind of the order of a 10-year event is appropriate.

(x) Both the normal freeboard and minimum freeboard computation follow the same procedure except that the significant wave height in equation in step (vii) for normal freeboard should be replaced by the design wave height which corresponds to the average of the highest 10 percent of the waves which is 1.27 times the significant wave height and maximum expected wind values should be used.

Also for normal freeboard the most severe winds are considered - in terms of velocity, duration, direction, and seasonal distribution that are reasonably characteristic of the region in which the reservoir is located. This includes the results of meteorological studies and probability analyses of recorded wind data. The values selected should exceed 100-year winds determined by probability analyses and generally should exceed maximum recorded winds.

3.1.4 USBR - ACER Technical Memorandum no. 2 - Freeboard Criteria and guidelines for computing Freeboard allowances for storage dams (1992)

The Bureau of Reclamation's freeboard policy as per this technical memorandum for new and existing concrete and embankment dams is briefly as follows:

New concrete dams. - Dams made with conventional concrete or roller compacted concrete, and any other types of dams that can resist the erosive action of temporary overtopping flow should be designed so that the top of the non-overflow section of the dam is coincident with the maximum water surface (MWS) elevation. The standard 3.5-foot (1.1 m) high solid parapet wall entirely above the dam top elevation of the non-overflow section provides for minimum freeboard in the event of the probable maximum flood (PMF). Due to the ability of concrete dams to resist erosion, this is ordinarily the only type of freeboard necessary to consider. Exceptional cases may point to a need for more freeboard, depending on the anticipated wave height or other factors such as erodibility of the downstream foundations and abutments.

New embankment dams. - Freeboard should be determined for new embankment dams above various water surface elevations in order to select a design crest elevation that adequately protects the embankment from the full range of wind and flood loading conditions. The design crest elevation should be the highest that would result from freeboard obtained corresponding to all such water surface elevations considered.

Freeboard requirements. - Although the freeboard requirements can be defined for both concrete and embankment dams, criteria for the computation of freeboard for embankment dams has been given more importance in this memorandum as they are more vulnerable to failure due to overtopping. Unlike concrete dams, embankments are erodible and may fail if adequate allowance for freeboard is not available.

(1) Freeboard criteria at Maximum Water Surface. - When the reservoir is at Maximum Water Surface, the minimum freeboard should be the greater of: (a) 3 feet (0.9 m), or (b) the sum of the setup and run-up that would be generated by the average winds that would be expected to occur during large floods. If the reservoir or watershed is very large in comparison to the size of the storm, the wind events that occur when the water surface is near maximum may be statistically independent of the storm that created the flood. In this case, a typical wind of not less than 10 percent exceedance probability should be used to compute a run-up for minimum freeboard.

(2) Normal water surface freeboard criteria. - When the reservoir is at the normal water surface i.e. at top of

joint-use capacity or top of active conservation capacity, a normal freeboard should be worked out that protects the dam against wind-generated waves that would occur due to the highest sustained velocity winds that could reasonably occur (e.g. 60-100 mile/hr. (95-160 km/hr.)).

(3) Intermediate water surface freeboard criteria. - When the reservoir is at an intermediate elevation, that is, an elevation between the Maximum Water Surface and the Normal Water Surface or top of joint-use or active conservation capacities, an intermediate freeboard requirement should be determined that has a remote probability of being exceeded by any combination of wind-generated waves and water surfaces occurring simultaneously.

The recommended approach to perform an embankment dam freeboard analysis is to start by choosing 3 feet (0.9 m) of minimum freeboard above the maximum water surface and then check to see if such a crest elevation would satisfy normal and intermediate water surface requirements. Allowances for camber and additional factors are added after the 3 feet of minimum freeboard is found satisfactory. Figure 3-2 shows the flow chart for freeboard analysis recommended in this memorandum.

These two checks can be performed initially to see if the 3 feet of minimum freeboard is sufficient or otherwise to prevent overtopping. For new USBR dams these two checks are the first steps of the analysis.

If the dam design fails either of these two checks then a probabilistic analysis should be used to evaluate the adequacy of 3 feet or more minimum freeboard. The probabilistic method is a more rigorous analysis, which has been described in this technical memorandum that evaluates the probability of overtopping various target crest (dam top) elevations from all possible water surfaces below the maximum.

The checks to evaluate the adequacy of 3 feet (0.9 m) of minimum freeboard are as under:

1) The first check is used to see if the dam top (crest) elevation assumed with the above minimum freeboard is adequate to protect the dam from overtopping should waves build up from a 100 mile/hr. (160 km/hr.) wind velocity while the reservoir is at normal water surface elevation.

2) The second check is used to see if the dam top (crest) elevation assumed with the above minimum freeboard is adequate to protect the dam from overtopping in the event of PMF and winds that would typically occur during the time period equal to the duration that the reservoir water surface is near maximum.

For checking the intermediate freeboard the information required includes:

- Reservoir Elevation versus Time graph derived from routing the PMF.
- Hourly Probability of the Wind (P_{w_H}) versus Wind Velocity derived from the analysis of wind data part of the probabilistic method.

Horizontal lines are drawn at the intermediate reservoir levels considered say at about 2 and 4 feet (0.6 and 1.2 m) or at any other suitable reservoir level below the Maximum Water Surface across the Reservoir Elevation versus Time graph. The duration that the reservoir is at a particular assumed intermediate reser-

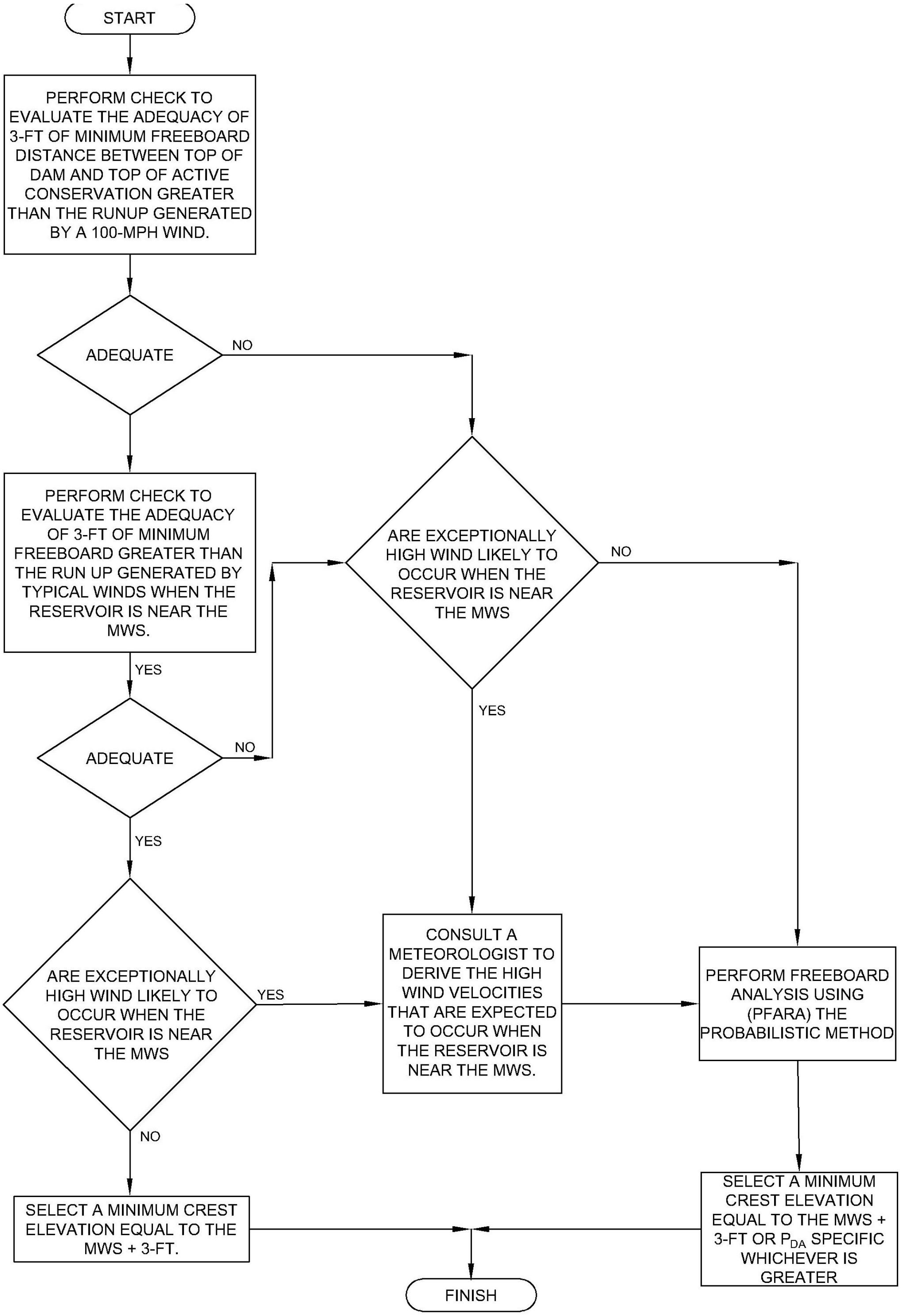


Figure 3-2: Flowchart for freeboard analysis to protect new embankment dams from overtopping failure due to wind generated waves

voir elevation, say 2 feet (0.6 m) below the Maximum Water Surface while passing the flood, is equal to the length of the horizontal line drawn at that intermediate reservoir elevation in the above graph prepared between Reservoir Elevation and Time. Inverse of this duration is equal to the hourly probability of the largest wind event that may typically occur while the reservoir is within 2 feet of the Maximum Water Surface.

The wind velocity is taken from the P_{w_H} versus Wind Velocity curve, which is required to be prepared as per procedure given subsequently under the head - Analysis of existing wind data.

This wind velocity and the reservoir fetch are used to determine the freeboard above the intermediate reservoir level considered i.e. 2 feet below the Maximum Water Surface for this case. Similar procedure can be adopted for estimating the freeboard at any other intermediate reservoir elevation.

If the dam design passes the above checks ,then no other method needs to be used to calculate freeboard especially in cases in which is no reason to believe that exceptionally high wind velocities (those not necessarily typical) would be blowing. On the other hand, if the dam design fails either of the above checks, then the probabilistic method should be used as described in the Technical Memorandum. It may still show that a dam crest elevation say 3 feet above the Maximum water surface may provide for an acceptable design probability. In summary, the scope of a freeboard analysis for new Reclamation embankment dams can be described by the flow chart given in Figure 3-2.

To preclude development of seepage caused by the Maximum reservoir water surface, the top of the impervious zone must be designed so that after settlement it is at the elevation of the Maximum Water Surface plus wind setup (but not run-up), with wind setup calculated from the winds associated with the largest flood events or typical winds of not less than 10 percent exceedance probability, whichever is greater

Existing concrete and embankment dams-

Freeboard for an existing concrete dam is not as critical as it is for an embankment dam because concrete is not likely to get washed away if the dam is overtopped. In the case of a concrete dam, failure will depend on the ability of the abutments and foundation to survive the force of the water flowing over the concrete dam. Although some existing concrete structures may be in such a poor condition that overtopping may cause extensive damage, a failure that would threaten the safety of life and property downstream is not likely on account of this damage. However, the geology of the dam site should be carefully examined by engineers and geologists to make a judgment on the potential for erosion or plucking of the materials. Fault zones and other types of discontinuities, weathered rock, friable or weakly cemented material, and soft intact rock are some geological features that may not survive well during overtopping. If erosion of the abutments or foundation leads to undercutting of the concrete structures, failure may result.

A Safety of Dams evaluation may be performed when it is decided that insufficient freeboard would lead to dam failure.

The freeboard requirements for an existing dam may be different than the requirements for a new dam. The incremental costs for raising a new dam a few feet during design would be minimal compared to the costs for doing the same to an existing dam.

Expenses would be much greater for a modification to an existing dam in terms of design data acquisition, design time, contracting, construction mobilization and unit price of materials. The option of changing the spillway design to accommodate larger floods may be quite costly for an existing dam while it may have little impact on the cost for a new dam under design. Thus, a risk cost analysis could provide a basis for selecting an amount of freeboard different for an existing dam from the amount of freeboard that would be acceptable for new dams. Such a study may be needed before a decision to raise an existing dam would be warranted.

The decision to modify an existing dam to provide more freeboard should consider the above factors in addition to the policies described for the freeboard requirements for new embankment dams (minimum, normal and intermediate freeboard).

The evaluation of existing dams also needs to take into account conditions that may have changed since the initial freeboard design determination. For example, the risk of malfunction of spillway and outlet works should be better known than at the time of original design because of maintenance and operating experience. When assessing the risk of malfunction, known limitations to gate operation should be considered as well as improvements in mechanical and electrical features or added provisions for skilled attendance during periods of operation. Because foundation and embankment settlement are likely to have occurred, the addition of a parapet wall may be a feasible method of providing freeboard in some embankment dam cases.

Parapet walls. – As per this memorandum a standard 3.5-foot (1.1 m) high parapet wall can provide the freeboard required for concrete dams. This wall is intended to keep waves from washing over the dam during high reservoir water levels.

Use of parapet walls to provide freeboard allowances for embankment dams may be considered on a case-by-case basis. The parapet wall ordinarily only replaces the portion of the freeboard needed to prevent overtopping from wave run-up but, in some cases, can be used to retain the uppermost flood storage of the extreme flood events for a very short time. When used, the following safeguards must be met:

- The parapet wall should be adequately tied into the impervious zone and proper zoning provided to prevent piping.
- Future foundation and embankment settlement that would adversely affect the structural integrity of the parapet wall must be provided for in construction sequencing or the design.
- Consideration must be given to hydrostatic and hydrodynamic (wave) loads, drainage off the crest around or through the wall, adjoining and sealing the wall units together with each other and each end of the dam, maintenance and aesthetics.

Methodology for calculating freeboard – Some of the important aspects recommended in this memorandum are given below:

Fetch Calculations

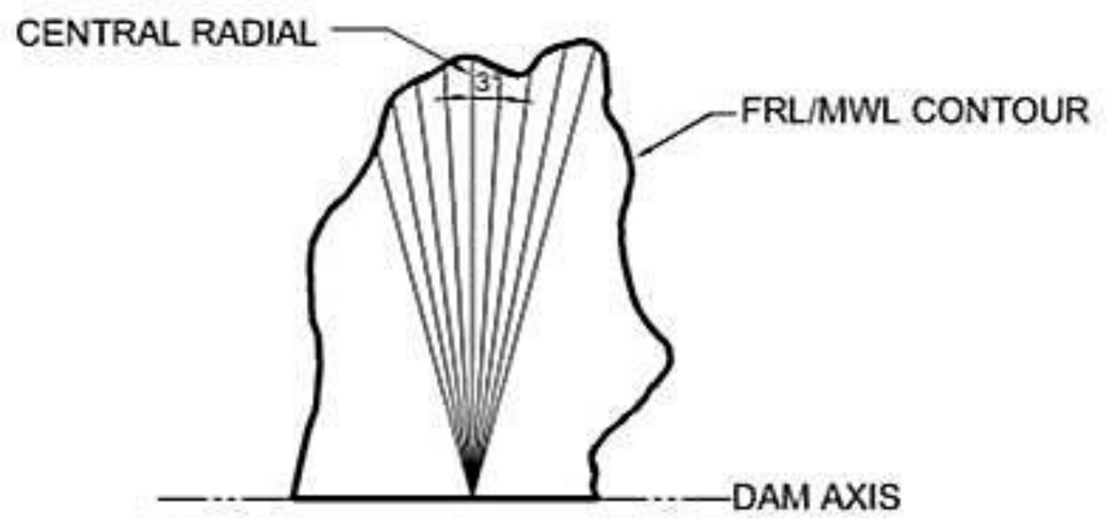
The procedure recommended for estimating the fetch over an inland reservoir having an irregularly shaped shoreline consists of constructing nine radials from the point of interest at 3 ° intervals and extending these radials until they first intersect the shoreline again on the opposite side of the reservoir (see Figure.3-3). The length of each radial is measured and arithmetically averaged. While 3° spacing of the radials is recommended, any other small angular spacing could be used. This calculation should be performed for several directions (of the central radial) approaching the dam, including the direction where the central radial is normal to the dam axis and also the direction where the 24° total spread results in the longest possible set of radials.

For each fetch calculated, the angle of the central radial with respect to a line normal to the dam axis should be determined. This angle is used to adjust the wave height considering that the wave may approach the dam from a less severe direction,

In earlier wave prediction methodologies, effective fetch was considered. Subsequently it has been seen that if the "effective fetch" is used for freeboard calculations, then the wave height will be underestimated. Thus, effective fetch has not been used with the curves of this memorandum. This is one major departure from the earlier USBR Memorandum of the year 1981.

Simultaneous Occurrence of all Freeboard Components

The possibility that some combinations of the components of freeboard occurring simultaneously is extremely low. Maximization of all components and adding them together to determine total freeboard requirements is unreasonable. Only those components which can reasonably occur simultaneously for a particular water surface elevation should



Fetch $= \sum_{n=1}^{9} (\boldsymbol{Length\ of\ Radials}) / (\boldsymbol{Number\ of\ Radials})$

$=$ (4.00+4.15+4.25+4.35+4.35+4.25+4.25+4.50+4.55)/9 = 4.20 miles

Figure 3-3: Fetch Calculation

be combined. The design crest elevation of the dam should be established to accommodate all combinations of water surface and wind occurrences with other freeboard components that are deemed reasonable. The design crest elevation excludes camber, road surfacing, and associated crown. It is highly unlikely that maximum winds will occur when the reservoir water surface is at its maximum elevation resulting from routing the PMF. Computations of wind-generated wave height and wind setup for intermediate freeboard should incorporate the probability of combined occurrences of reservoir elevation and duration and wind velocity and duration. Consideration should be given to the shape of the reservoir elevation versus time curves during flood events. Although the maximum reservoir elevation associated with many flood events may be expected to last only a few hours, a reservoir elevation close to but lower than the maximum may have a much longer duration. The freeboard analyst must examine many combinations of intermediate reservoir elevations and wind events generating wave run-up and setup such that a minimum crest elevation may be determined that would protect the dam from all possible wind and flood events.

Probabilistic method. - The probabilistic method for computing freeboard requirements was recommended for use in this memorandum for new dams whenever the criteria for the simplified approach is not satisfied.

The probabilistic freeboard analysis involves two basic steps. The first step is, in effect, the derivation of the cumulative probability distribution of a sum of two independent random variables, generically Z = X + Y, where in this case X is the water surface elevation corresponding to a flood level (expressed as a fraction of the PMF) and Y is the wind-induced incremental height (due to wave run-up and wind setup). The type of calculation required, known as convolution, generally proceeds as follows: the range of one of the random variables (in this case, the water surface elevation) is divided into non-overlapping increments, and the probability of not exceeding a given value of the sum, denoted by Z (representing, in this case, a "target" crest elevation, is found by multiplying non-exceedance probabilities, one for each increment (of the water surface elevation). The probability of exceedance of a given crest elevation, denoted here as p = Prob {Z ≥ z}, is expressed with reference to a 1-hour interval, randomly chosen within a year.

The second step is to convert the hourly exceedance probability of a given design freeboard level z into an annual probability of exceedance. A given freeboard level is not exceeded in a 1- year period if it is not exceeded in any one of the non-overlapping 1-hour segments that make up 1 year.

The calculated "hourly risks" p = Prob {Z ≥ z}, depends on the assumed probability distributions of the input random variables "X" and "Y." In particular, the range of water surface elevations has a prescribed minimum and maximum value, the latter corresponding to the PMF whose exceedance probability may be arbitrarily set at 10^{-4} per year for relative analysis purposes. In light of these assumptions, it is desirable that a sensitivity analysis be done to determine the relative sizes of contributions to the "hourly risks" p (or "annual risks" P") from different flood levels, i.e., floods corresponding to different fractions of the PMF. It also makes sense to consider the impact on the overtopping risk due to floods that are "multiples of the PMF," associated with mean annual occurrence rates below 10^{-4}.

It should be noted that the use of this method is greatly facilitated by the use of computers with programs already developed by the Bureau of Reclamation at their Denver Office.

The steps envisaged are briefly outlined below.

a) Analysis of existing wind data - This envisages:

1) Collection of data from the Wind Energy Resource Atlases published by Battelle Pacific Northwest Laboratory pertaining to USA.

2) Converting the wind data to probabilities - The tables of wind persistence from Battelle list the number of occurrences that a given wind velocity has been exceeded for a selected number of consecutive hours. By converting the "number of occurrences" to "number of hours" and dividing by the total number of hours of the period of record, the value "P_{W_H}" the probability of the wind exceeding a given velocity for a specific number of hours, is derived.

3) Transposition of the probabilities to the reservoir site

4) Preparation of wind event curves - The probability of wind exceeding a given velocity P_{W_H} for 1, 2, 3, 4, and 5 consecutive hours at the reservoir is plotted for each wind velocity. The data points are plotted on semi-logarithmic paper, and a best fit curve is drawn for each velocity. Each curve represents the probability of the wind exceeding a specific velocity for a selected duration P_{W_H} during any wind event.

5) Over water correction for adjusting over land to over water velocities.

6) Minimum wind duration to reach maximum wave heights - Both wind duration and fetch distance can limit the height of waves caused by a given wind velocity. Waves are assumed to grow continuously under the action of the wind as they move along the fetch. Given the minimum fetch distance needed, the waves will reach a maximum height that can be sustained by the wind velocity. There will be no further increase in wave height regardless of how long the wind blows or how much the fetch exceeds the minimum needed. Conversely, given a limited fetch distance, the maximum wave height for a particular velocity will not be reached because the waves will collide with the shoreline or dam before reaching maximum height.

The duration needed for a given wind velocity to generate the highest waves is designated the minimum duration. The value of fetch is used to obtain minimum durations for the wind velocities (adjusted to overwater velocities) corresponding to the wind event curves. The minimum duration can be computed as:

$$t\ min = 1.912\ (F^{0.66} / V^{0.41})$$

where,

t min = Minimum duration required to build up the maximum waves (hours)

F = Fetch (miles)

V = Wind Velocity over water (mile/hr)

The minimum durations are then plotted on the respective wind event curves. The probability of each velocity being exceeded for the minimum duration needed to produce a maximum wave height is the ordinate corresponding to the minimum duration plotted on the wind event curve.

7) Wind event probabilities - A curve joining the minimum wind durations plotted on the wind event curves represents the probability of a selected overwater wind velocity being exceeded for the minimum duration needed to produce its maximum wave. For ease in determining the wind velocity likely to occur for a minimum duration during a given reservoir water surface event, a curve of probability of wind velocity being exceeded (P_{W_H}) versus wind velocity (overwater) should be drawn on semi-logarithmic paper. Values of (P_{W_H}) and their respective overland (converted to overwater) velocities corresponding to the minimum duration for each velocity should be used.

b) Analysis of flood data - Reservoir flood storage between the top of active conservation or joint-use capacity and the MWS is divided into intervals. Discrete probabilities can be computed for each interval based on flood data. When the wind events are added to these reservoir events, wave run-up, and wind setup determine the target crest elevations.

1) Reservoir events. - All intermediate reservoir events between the top of active conservation or joint-use capacity and the MWS are used in this probabilistic approach. The PMF, 4,000, 1,000, 400, and 100-year flood events are routed so that interpolations can be made to obtain the durations of any reservoir elevation.

2) Duration of reservoir water surface. - Twenty or so intermediate reservoir water surface elevations are selected between the top of active conservation or joint-use capacity and the MWS. Durations that the water is at or above each elevation for each flood are determined from the elevation versus time information of the flood routings. These durations are plotted versus their annual probability (1 / the flood mean return period) on semi-logarithmic paper. Lines are drawn through points representing equal reservoir elevations. These lines are divided up into 10 or so increments and the probability times the average duration of each increment is summed to obtain DURELV, the amount of time (in hours) that the reservoir can be expected at or above each elevation each year.

3) Probability of reservoir water surface intervals. - The hourly probability of the reservoir exceeding the given elevation (P_{R_H}) is calculated by:

$$\boldsymbol{P_{R_H}}=\frac{\boldsymbol{DURELV}}{\mathbf{(24)x(365)}}$$

Where,

P_{R_H}, = the hourly probability of exceeding the given reservoir elevation.

DURELV = the total duration that the reservoir would be expected at or above the given elevation in any year (hr.)

The hourly probability of being within a reservoir interval $P_{(RR(interval))_H}$ is the difference between the P_{R_H} of the two bounding reservoir elevations or:

$$P_{(RR(interval))_H} = \frac{DURINT}{(24)x(365)}$$

where,

$P_{(RR(interval))_H}$ = Hourly probability of the reservoir being within a certain interval of two elevations.

DURINT = Time that the reservoir would be expected to be within the certain interval any year. The difference between the DURELV of each reservoir elevation bounding the interval (hr.).

c) Wind Effects on water

Wave Height - Wind-generated waves in large bodies of water are not uniform in height but consist of spectra of waves with various heights. A well-defined relationship exists between the significant wave height (H_s) and the heights of the other waves in the spectrum. The relationship is shown in Table 3-4. From this tabulation, it can be seen that Hs represents the average height of the highest one-third of the waves in a given spectrum. Likewise, the average wave height of the highest 10 percent of the waves in a given spectrum is 1.27 H_s and the average wave height of the highest 1 percent of the waves in a given spectrum would be approximately 1.67 H_s.

The maximum wave height ratio to be used to compute wave run-up for normal free freeboard above normal water surface elevation should be selected on the ability of the crest and downstream slope to withstand overtopping by wave action. When the crest and downstream slope are adequately protected against erosion or will not slough or soften excessively, or when public traffic will not be interrupted, a wave height equal to the average height of the highest 10 percent of the waves (1.27 x height of significant wave) should be used to compute run-up. A wave height equal to (1.67 x height of the significant wave) should be used if overtopping by only an infrequent wave is permissible.

The height of significant wave due to each wind event can be determined from the relationship

Table 3-4: Common wave height relationships

Percent of total number of waves in series averaged to compute specific wave height (H)	Ratio of specific wave height, H, to average wave height, H_{ave} (H/H_{ave})	Ratio of specific wave height, H, to significant wave height Hs (H/Hs)	Percent of waves exceeding specific wave height, H
1	2.66	1.67	0.4
5	2.24	1.40	2
10	2.03	1.27	4
20	1.80	1.12	8
25	1.71	1.07	10
30	1.64	1.02	12
33.33	1.60	1.00	13
40	1.52	0.95	16
50	1.42	0.89	20
75	1.20	0.75	32
100	1.00	0.62	46

$$H_s = 0.0177\ (V)^{1.23}\ (F)^{0.5}$$

Where,

H_s = Height of significant wave in feet

V = Wind velocity, in miles per hour

F = Fetch, in miles

Wave heights for waves computed for fetches that are not normal to the dam axis (i.e. when the central radial is not normal to the dam axis) should be reduced according to a factor derived from figure 8 of the memorandum

1) Wave length and wave period. - The deep water wave length (L) in feet and the wave period (T) in seconds can be computed by the relationships:

$$L = 5.12\,T^{2}$$

$$T = 0.559\,(0.589\,(V)^{1.23}\,(F))^{0.33}$$

where V and F have been defined earlier.

It may be assumed that the wave period T is the same for all wave heights in a given wave spectrum.

Most dams have relatively deep reservoirs compared to the wind-generated wave length, and the wave is unaffected by the reservoir floor. The above equations for wave height, wave period, and minimum duration are valid when the water is deeper than one-half of the wave length. If reservoir depth becomes a limiting factor, different relationships for shallow water waves should be used. Wave height, wave period, and minimum duration for shallow water waves can be obtained from figures 9-18 of the memorandum.

2) Wave run-up. - If a deep-water wave reaches a sloping embankment without major modification in characteristics, the wave will ultimately break on the embankment and run up the slope to a height governed by the angle of the slope, the roughness and permeability of the embankment surface, and the wave characteristics. Wave run-up, R, is the vertical difference between the maximum level attained by the rush of water up the slope and the still water elevation. Run-up, from a wave, on an even embankment with a riprap surface is given by:

$$R = \frac{H}{0.4 + \left(\frac{H}{L}\right)^{0.5} \cot\theta}$$

where,

R= Vertical component of wave run-up in feet

H= Wave height in feet

L= Wave length in feet

θ= Angle of the dam face from horizontal

This equation should be used only for dam slopes of 5 (H): 1 (V) or steeper.

For embankment dams with soil-cement, rounded cobbles and boulders for riprap, or other protective surfaces not as rough as irregular dumped angular riprap, the run-up computed by the above equation should be multiplied by a factor of up to 1.5 (for the smoothest embankment surfaces, such as soil cement), depending on the relative smoothness of the surface.

The above equation should however not be used for computing run-up for rock fill dams. Figure 19 of the memorandum can be referred for the same.

3) Wind setup. - Wind blowing over a water surface exerts a horizontal shear force on the water, driving it in the direction of the wind. In an enclosed body of water, the wind effect results in a rise in the water level at the leeward end of the fetch. This effect is termed "wind-tide" or "wind setup".

Wind setup in feet, S, is computed as follows:

$$S = V'^2 F / 1400\, D$$

V= Design wind velocity over water in miles per hour

F= Wind fetch in miles

D= Average water depth in feet

The value of D should be a reasonable approximation of the average depth along the fetch length, with more emphasis given to depths with-

board; namely, ACER Technical Memorandum No. 2 (1992). However, the Shore Protection Manual has been updated and now is called the Coastal Engineering Manual (CEM), numbered EM-1110-2-1100.

Part II and Part VI of the CEM are applicable to freeboard computations, particularly chapter 2 of Part II (dated August 1, 2008) for wave characteristics and chapter 5 of Part VI (dated September 28, 2011) for run-up and setup calculations, as well as the USACE freeboard analysis and compu-

<table>
<tr><th>Type of Freeboard</th><th colspan="2">Approach to Freeboard Analysis</th></tr>
<tr><td rowspan="2">Minimum</td><td colspan="2">Select a design crest elevation - Higher of:</td></tr>
<tr><td>MRWS + 3 feet</td><td>MRWS + run up and setup from a wind velocity exceed 10% of the time.</td></tr>
<tr><td>Normal</td><td colspan="2">NRWS + run up and setup from a 100 mile per hour wind velocity</td></tr>
<tr><td rowspan="2">Checks for Intermediate</td><td colspan="2">Design crest elevation is adequate for run up and setup during the IDF when reservoir is within 2 feet of the MRWS</td></tr>
<tr><td colspan="2">Design crest elevation is adequate for run up and setup during the IDF when reservoir is within 4 feet of the MRWS</td></tr>
<tr><td colspan="3">Note: MRWS = Maximum Reservoir Water Surface, NRWS = Normal Reservoir Water Surface, IDF = Inflow Design Flood.</td></tr>
</table>

Figure 3-4: Freeboard Calculations

in a few miles of the location for which the setup is being computed. The direction of fetch is taken as that of the central radial used in computing the fetch.

3.1.5 USBR – Design Standards No.13 – Embankment Dams, Chapter 6: Freeboard, September 2012

The U.S. Army Corps of Engineers (USACE) has carried out a large amount of research on wave height determination and wave run-up on embankments. The results of that research are contained in many of their references and guidelines. The Shore Protection Manual, fourth edition, published in 1984, was the basis of previous Bureau of Reclamation reference on free-

tations for estimating the probability of overtopping. The CEM updates were used as the basis of this version of the Freeboard Design Standard. The recommended freeboard calculations in the form of a flow chart is at Figure 3-4.

The main provisions are briefly given below:

a) Fetch – This Standard recommends the same procedure for estimating the fetch as recommended earlier in USBR Technical Memorandum no.2 (Revised 1992 edition). The procedure consists of constructing nine radials from the point of interest at 3-degree intervals and extending these radials until they first intersect the shoreline again on the opposite side of the reservoir (see Figure 3-3). The length of each radial is measured and arithmetically averaged. While 3-degree spacing of the radials is recommended, any other small angular spacing could also be used. This calculation should be performed for several directions (of the central radial) approaching the dam, including the direction where the central radial is normal to the dam axis and also the direction where the total spread results in the longest possible set of radials.

For each fetch calculated, the angle of the central radial with respect to a line normal to the dam's axis should be determined. This angle is used with an appropriate reduction factor to adjust the run-up, considering that the wave may approach the dam from a less severe direction.

The earlier concept of effective fetch was not recommended by USBR (as recommended in USBR Technical Memorandum no.2 -1981 edition) as it was apprehended that the waves will be underestimated. Thus, effective fetch is not be used with the curves of this standard.

b) Probabilistic Method – The Technical Memorandum No. 2 (1992 edition) had recommended the use of the sophisticated probabilistic analysis for freeboard computations.

The method used to compute a probability distribution function for elevations for the dam crest as the result of combining the probabilities of floods to produce reservoir levels below the crest and the probabilities of wind to generate waves that caused run up and setup to reach the crest elevation.

A computer program PFARA (which stands for "Probabilistic Freeboard and Riprap Analysis") was developed to perform these computations.

This complicated procedure was however not used. The simpler method presented in this design standard was used instead, which also gives good results.

PFARA is however still used in freeboard and riprap analysis but primarily only for the analysis of wind data and the derivation of design wind events. This part has been covered in this standard. The program uses site-specific data to produce a probability distribution of wind velocity over water.

c) Analysis of existing wind data - This envisages:

1. Collection of data from the Wind Energy Resource Atlases published by Battelle Pacific Northwest Laboratory pertaining to USA.
2. Converting the wind data to probabilities - The tables of wind persistence from Battelle list the number of occurrences that a given wind velocity has been exceeded for a selected number of consecutive hours. By converting the "number of occurrences" to "number of hours" and dividing by the total number of hours of the period of record, the value "P_{W_H}" the probability of the wind exceeding a given velocity for a specific number of hours, is derived.

   This approach is similar to that in USBR Technical Memorandum No. 2 (1992 edition)

   A typical probability distribution is at Figure 3-5.
3. Transposition of the probabilities to the reservoir site.
4. Preparation of wind event curves: The probability of wind exceeding a given velocity P_{W_H} for 1, 2, 3, 4, and 5 consecutive hours at the reservoir is plotted for each wind velocity. The data points are plotted on semi-logarithmic paper, and a best fit curve is drawn for each velocity. Each curve represents the probability of the wind exceeding a specific velocity for a selected duration P_{W_H} during any wind event.

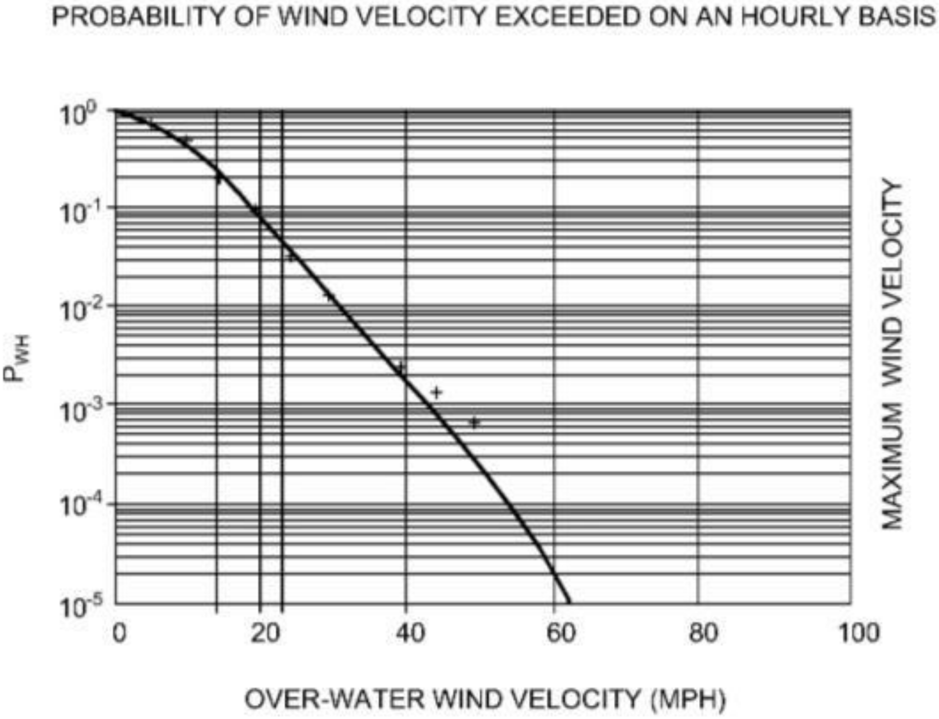


Figure 3-5: Wind Probability Curve

Over water correction for adjusting over land to over water velocities Figure-3-6 given below is used for the purpose.

5. Minimum wind duration to reach maximum wave heights - The duration needed for a given wind velocity to generate the highest waves is designated the minimum duration. The value of fetch is used to obtain minimum durations for the wind velocities (adjusted to overwater velocities) corresponding to the wind event

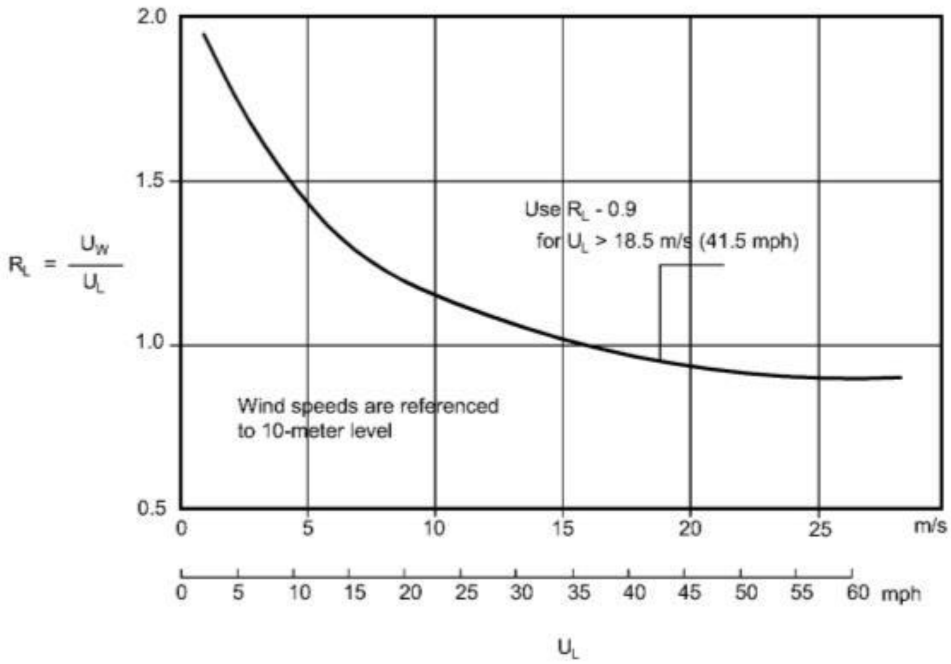


Figure 3-6: Ratio of wind speed over water to wind speed over land as a function of wind speed over land

curves. The minimum duration can be computed as:

$t\ min = 1.87\ (F^{0.67} / VMPH^{0.34})$

Where,

t min = Minimum duration required to generate the maximum wave height (hours).

F = Fetch (miles)

VMPH = Wind Velocity (mile/hr.)

The minimum durations are then plotted on the respective wind event curves. The probability of each velocity being exceeded for the minimum duration needed to produce a maximum wave height is the ordinate corresponding to the minimum duration plotted on the wind event curve.

Wind event probabilities - A curve joining the minimum wind durations plotted on the wind event curves represents the probability of a selected overwater wind velocity being exceeded for the minimum duration needed to produce its maximum wave height. For ease in determining the wind velocity likely to occur for a minimum duration during a given reservoir water surface event, a curve of probability of wind velocity being exceeded on an hourly basis (P_{w_H}) versus wind velocity (overwater) should be drawn on semi-logarithmic paper. Values of (P_{w_H}) and their respective overland (converted to overwater) wind velocities corresponding to the minimum duration for each velocity should be used.

6. For normal freeboard above normal reservoir surface elevation, a wind velocity of 100 mile/hr. over water has been recommended in this standard for embankment dams of USBR. This can be calculated from the formulas/curves given in the standard or can be directly read from the figure 6.2.2-1 given in the standard.
7. For minimum freeboard above maximum water surface elevation, a wind velocity with 10 % probability of exceedance on an hourly basis was recommended which was to be taken from the probability curve at Figure 3-5.

d) Wind Effects on Water

1. Wave Height

Wind-generated waves are not uniform in height, but they consist of a distribution of waves with various heights. The significant wave height (H_s) is defined as the average of the highest one-third of the waves in a wave field. The fetch-limited significant wave height (in feet) is given by:

$H_s = 0.0245\ F^{1/2}\ VMPH\ (1.1+0.0156\ VMPH)^{1/2}$

Using a Rayleigh distribution, other statistical wave height measures can be estimated from the significant wave height. For details see Table 3-4. It can be seen that the average wave height of the highest 10 percent of the waves is 1.27 H_s and the average wave height of the highest 1 percent of the waves in a given spectrum is approximately 1.67 H_s.

The wave height statistic used to compute wave run-up should be selected based on the ability of the crest and downstream slope to withstand overtopping by wave action. When the crest and downstream slope are adequately protected against erosion or will not slough or soften excessively, or when public traffic will not be interrupted, a wave height equal to the average height of the highest 10 percent of the waves (1.27 x height of significant wave) should be used to compute run-up. A wave height equal to 1.67 x height of the significant wave, should be used if overtopping by only an infrequent wave is permissible.

Wave heights for fetches that are not normal to the dam axis should be reduced according to a factor derived from Figure 3-7. Just as wave heights in a wave field are not uniform, there is also a distribution (spread) in wave directions. The significant wave height is multiplied by the reduction factor to obtain a reduced significant wave height for design.

2. Wave Length and Wave Period

The deep water wave length L (in feet) and wave period T (in sec.) can be computed by the relationships:

$$L = \frac{gT^2}{2\pi} = 5.12\,T^2$$

$$T = 0.464\ F^{1/3}\ VMPH^{1/3}(1.1 + 0.0156VMPH)^{1/6}$$

Wave periods are normally distributed about the peak period for locally generated waves. It may be assumed that the wave period, T, is the same for all waves in the wave field.

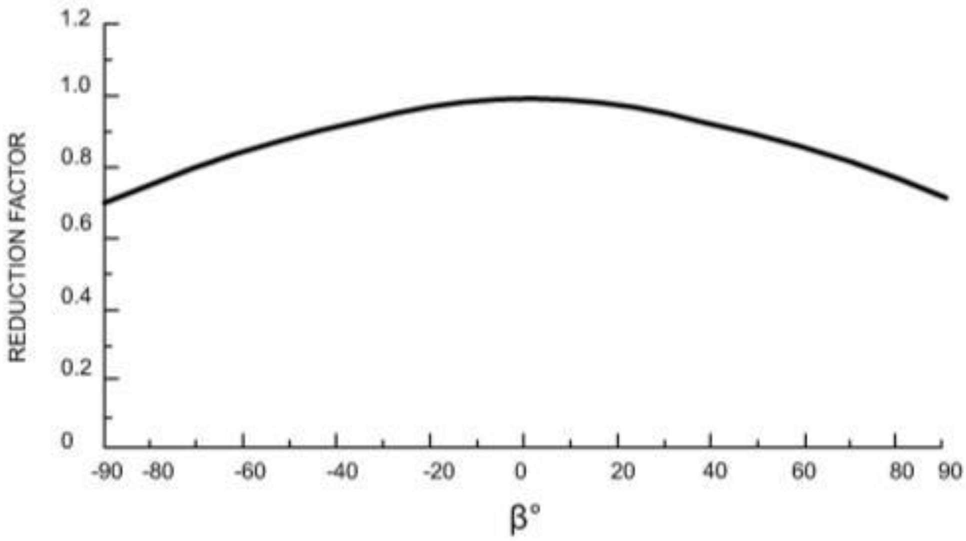
Most dams have relatively deep reservoirs compared to the wind-generated wave length and the wave is unaffected by the reservoir floor.

The above equations for wave height, wave period, and minimum duration are valid when the water is deeper than one-half of the wave length.

If reservoir depth becomes a limiting factor, different relationships for shallow water waves should be used. Wave height, wave period, and minimum duration for shallow water waves can be obtained from USACE EM 1110-2-1100 (Part VI).

3. Wave Run-up

When a deep-water wave reaches a sloping embankment without major modification in characteristics, the wave will ultimately break on the embankment and run up the slope to a height governed by the angle of the



β = The angle between the fetch and the dam axis (in degrees).
(0° is normal incidence and is commonly used to compute fetch, which is directly perpendicular to the dam axis).

Figure 3-7: Wave height reduction due to angular spread

Table 3-5: Surface roughness reduction factor (valid for 1 <ξ_p<3-4)

Type of Slope Surface	γ_r
Smooth, Concrete, Asphalt	1.0
Smooth block revetment	1.0
Glass (3 centimeters in length)	0.9 – 1.0
One Layer of Rock, diameter D, (Hs/D = 1.5 – 3.0)	0.55 – 0.60
Two or more Layers of rock, (Hs/D = 1.5 – 6.0)	0.50 – 0.55

slope, the roughness and permeability of the embankment surface, and the wave characteristics. Wave run-up, R, is the vertical difference between the maximum level attained by the rush of water up the slope and the still water elevation.

To compute the run-up, a surf similarity factor for peak wave heights, ξ_p is first computed from the following equation:

$$\xi_p = \frac{tan\alpha}{\sqrt{S_p}}$$

Where,

α = Slope angle of the upstream face of the embankment dam with the horizontal.

If the upstream slope of an embankment dam is 3(H): 1(V) then tan α = 0.33.

(Note: These equations used to compute run-up should be used only for dam slopes of 5(H):1(V) or steeper.)

S_p = The steepness of the peak waves which is computed as follows

$$S_p = \frac{H_s}{L} = \frac{2\pi}{g}\frac{H_s}{T^2}$$

H_s = Significant wave height (feet) of the incident waves.

L = Wave length (feet)

T = Wave period (seconds)

This results in,

$$\xi_p = \frac{2.26\,T\,(tan\alpha)}{\sqrt{H_s}}$$

The wave run-up R, is computed by the following equation:

$$R = H_s(A\xi_p + C)\gamma_r\gamma_b\gamma_h\gamma_\beta$$

Where,

R = Run-up on a relatively impermeable slope (i.e., the upstream slope of an embankment dam) (feet)

H_s = Significant wave height (feet)

ξ_p= Surf similarity factor (from the previous equations)

A, C = Coefficients dependent on ξ_p (see Table 3-6) and the probability of the runup (2 percent is used for freeboard and riprap calculations)

Table 3-6: Values for variables A and C of the runup equation

ξ_p Limits	A	C
$\xi_p \leq 2.5$	1.6	0
2.5< ξp<9	-0.2	4.5

$\gamma_r, \gamma_b, \gamma_h, \gamma_\beta$ = Reduction factors as under:

γ_r is a reduction factor to account for the roughness of the slope to be taken from Table 3-5 for use in the run-up equation above. For riprap, a value of 0.55 is suggested.

γ_b is a reduction factor for the influence of a berm. (γ_b = 1.0 for non-bermed profiles).

γ_h is a reduction factor for the influence of shallow-water conditions, where the wave height distribution deviates from the Rayleigh distribution. (γ_h = 1.0 for Rayleigh distributed waves).

γ_β is a reduction factor to account for a reduction in run-up due to the direction of the fetch relative to the dam axis. (See Figure 3-8)

Knowing the surf similarity factor for peak waves i.e. ξ_p , Table 3-6 is used to determine the variables A and C for use in the above equation for 2-percent run-up. (Average of the highest 2 percent of the run-ups, which is commonly used in CEM, as well as for freeboard and riprap analysis).

For run-up calculations on most embankment dam's freeboard analyses, γ_b and γ_h are set equal to 1.0. If shallow water wave distributions are greatly different than a Rayleigh distribution, or if there is a berm on the upstream slope, USACE EM 1110-2-1100 (Part VI) should be referenced for these other reduction factors.

4. Wind Setup: Wind blowing over a water surface exerts a horizontal shear force on the water, driving it in the direction of the wind. In an enclosed body of water, the wind effect results in a rise in the water level at the downwind end of the fetch. This effect is termed "wind tide" or "wind setup.

Wind setup in feet, S, is computed as follows:

$S = VMPH^2 F/1400D$

Where,

VMPH = Design wind velocity over water (mile/hr.)

F = Wind fetch (miles)

D = Average depth of water (feet)

The value of D should be a reasonable approximation of the average depth along the fetch length, with more emphasis given to depths within a few miles of the location for which the setup is being computed. The direction of fetch is taken as that of the central radial used in computing fetch.

3.1.6 IS-10635 - Free board requirements in Embankments dams and IS 6512 - Design of Solid Gravity dams

The Indian Standards viz. IS 10635 and IS 6512 have laid down the freeboard requirements/methodology for calculating the freeboard for Embankment and Concrete/Masonry Gravity dams respectively.

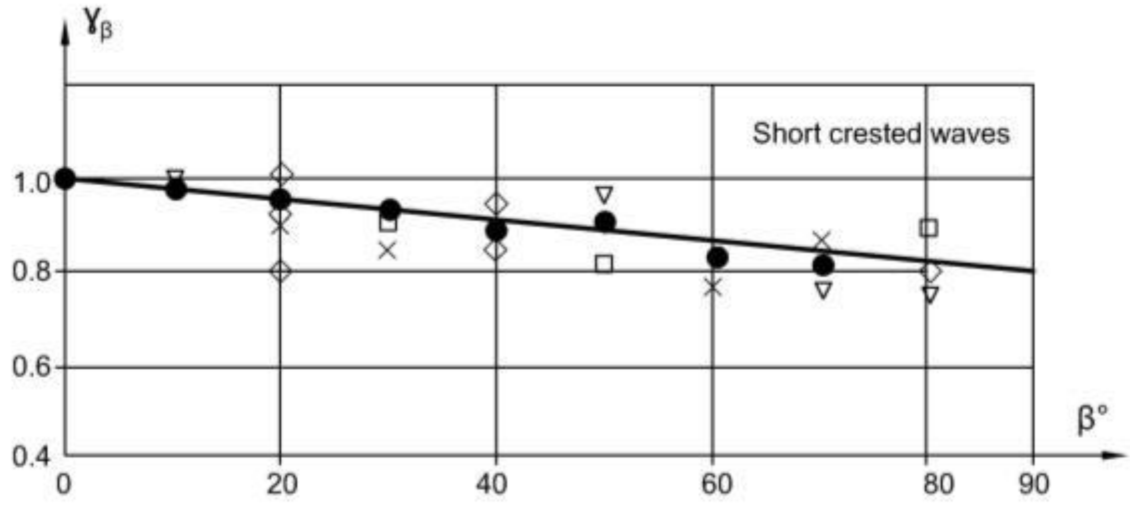


Figure 3-8: Influence of angle of incidence, β, and the directional spreading on runup on smooth slopes at Delft Hydraulics

The methodology given in the above IS standards is based on T. Saville's method.

The freeboard is calculated for the following conditions:

i) Normal Freeboard i.e. above Full Reservoir Level (FRL).

ii) Minimum Freeboard i.e. above maximum Reservoir Level (MWL) corresponding to design flood.

For Embankment dams the freeboard is calculated as the sum of wind set up and run up.

For Concrete/Masonry dams the freeboard is calculated as the sum of wind set up and $1\frac{1}{3}$ times the wave height corresponding to the reservoir elevation (FRL/MWL) under consideration.

The freeboard which gives the highest requirement of Top of Bund Level (TBL)/ Dam Top Elevation/ Dam Crest Elevation is finally adopted.

The normal freeboard should not be less than 2.0 m for embankment dams.

For calculating normal freeboard for embankment dams the design wave height is taken as 1.67 times the significant wave height which comes out to be the average height of the highest 1 % of the waves (The significant wave height is defined as the average of the highest one-third waves in the wave field). On the other hand for concrete/masonry dams the design wave height is taken as 1.27 times the significant wave height which comes out to be the average height of the highest 10 % of the waves while calculating the normal freeboard.

The minimum freeboard should not be less than 1.50 m for Embankment dams and not less than 1.00 m for Concrete/Masonry dams corresponding to design flood.

Where FRL and MWL are same the minimum freeboard shall be 2 m for Embankment dams.

For calculating minimum freeboard the design wave height is taken as 1.27 times the significant wave height for Embankment dams. For Concrete/Masonry dams this has not been clearly specified and is left to the discretion of the designer. Normally in concrete/masonry dams the design wave height is taken same as the significant wave height for minimum freeboard calculations.

Further IS 6512 on solid gravity dams prescribes that if the design flood is not same as Probable Maximum Flood (PMF) then the top of the dam shall not be lower than the reservoir level corresponding to PMF.

The 1.0 m high solid parapet wall on dam top is recommended to be provided in all dams as per the above IS codes. However it is not to be considered as part of the freeboard.

As regards fetch for use in the calculations of wave height and run up, effective fetch at the reservoir level under consideration is prescribed for use in the above Indian Standards. For calculating wind set up, the maximum straight line fetch at the reservoir level under consideration is to be used.

Wind velocity on land for the purpose of calculating the normal freeboard is taken from Fig.1 of IS 875 (Part 3): 1987 as the basic wind speed on land for 50 year return period for the region in which the dam falls. For minimum freeboard half to two-third the above wind velocity over land is taken.

These wind velocities over land are adjusted to over water velocities using the tables given for the purpose in the IS codes.

The actual freeboard calculations are carried out using various equations/figures/graphs given in the above IS codes.

3.2 Recommended Method for Freeboard Calculations for existing dams

Based on the above methods and practices, the following methodology is proposed for calculating freeboard for existing dams. As mentioned earlier, normal freeboard is calculated above FRL and minimum freeboard above MWL.

The freeboard which gives the highest requirement of Top of Bund Level (TBL)/ Dam Top Elevation/ Dam Crest Elevation is finally adopted.

For existing dams very often it is seen that the MWL for the revised design flood is higher than the original MWL for which the dam was designed. This results in reduction in the minimum freeboard, which needs to be checked for its adequacy.

In such cases it is recommended that where an existing solid u/s parapet wall exists on the dam top it may be considered as a part of the freeboard allowance as long as the revised MWL and the wind setup are sufficiently below dam top (by about 0.50 m or more) in Embankment dams.

In such cases for Embankment dams it is to be ensured that the top of the impervious core is above the revised MWL plus wind set-up. In case it is not so then a diaphragm wall of impervious materials or plastic concrete can be provided from the dam top.

In composite dams i.e. combination of Embankment dam and Concrete/Masonry gravity dam, the dam top will be generally governed by the freeboard calculations of the Embankment dam.

Detailed calculations of freeboard for the actual site-specific conditions are recommended to be carried out as outlined in the paragraphs below.

3.2.1 Normal Freeboard

1) It is the freeboard above the full reservoir level (FRL).

2) For Embankment dams the freeboard is calculated as the sum of wind set up and run up.

3) For Concrete/Masonry dams the freeboard is calculated as the sum of wind set up and $1\frac{1}{3}$ times the design wave height.

4) The normal freeboard should not be less than 2.0 m for embankment dams.

5) Wind velocity on land may be taken from Fig.1 of IS 875 (Part 3): 1987 as the basic wind speed on land for 50 year return period for the region in which the dam falls.

   Basic wind speed as per the above IS code is based on the peak gust velocity averaged over a short time interval of about 3 seconds and corresponds to mean heights above ground level in a open terrain.

6) Adjust the above wind velocity over land to wind velocity over water using Figure 3-6.

7) Calculate fetch at Full Reservoir level (FRL) as per the procedure given in Design Standard No.13 of USBR.

   The procedure consists of constructing nine radials from a point on the dam axis at 3-degree intervals and extending them till they intersect the FRL contour on the reservoir map (see Figure 3-3). The length of each radial is measured and arithmetically averaged to determine fetch.

   This calculation should be performed for several points on the dam axis with the central radial in different directions. Maximum fetch calculated should be used.

The earlier concept of effective fetch is not recommended as it results in the wave height being underestimated.

8) Calculate significant wave height using the formula,

$$H_s = 0.0245\ F^{1/2}\ VMPH\ (1.1+0.0156\ VMPH)^{1/2}$$

Where,

Hs = Significant wave height in feet.

F = Fetch at FRL in mile.

VMPH = Wind velocity on water in mile/hr.

This significant wave height is to be corrected for the inclination of central fetch with respect to the normal to dam axis using the reduction factor from Figure 3-7.

9) Design Wave Height

a) Embankment dams

Calculate Design Height (H_0) as under for normal freeboard purposes:

For protected embankments $H_0 = 1.27\ H_s$

For unprotected embankments $H_0 = 1.67\ H_s$

The embankment dam may be considered to be protected if the road on dam top is in good condition and the downstream slope is protected with turfing, surface drainage etc.

b) Concrete/Masonry dams

Take $H_0 = 1.27\ H_s$

10) Wave Length and Wave Period

The deep water wave length L (in feet) and wave period T (in sec.) can be computed by the relationships:

$$L = \frac{gT^2}{2\pi} = 5.12\ T^2$$

$$T = 0.464\ F^{1/3}\ VMPH^{1/3}(1.1+0.0156VMPH)^{1/6}$$

The above equations for wave height, wave period, and minimum duration are valid when the u/s reservoir depth is greater than one-half of the wave length.

If reservoir depth becomes a limiting factor, different relationships for shallow water waves should be used. Wave height, run-up, wave period and minimum duration for shallow water waves can be calculated as per USACE - CEM 1110-2-1100 (Part VI).

11) Wave Run-up

a) Embankment dams

First compute surf similarity factor for peak wave heights, ξ_p from the following equation:

$$\xi_p = \frac{tan\alpha}{\sqrt{S_p}}$$

Where,

α = Slope angle of the upstream face of the embankment dam with the horizontal.

If the upstream slope of an embankment dam is 3(H): 1(V) then tan α = 0.33.

(Note: These equations for computing run-up should be used only for dam slopes of 5(H):1(V) or steeper.)

S_p = The steepness of the peak waves which is computed as follows

$$S_p = \frac{H_0}{L} = \frac{2\pi}{g}\frac{H_0}{T^2}$$

H_0 = Design wave height (feet) of the incident waves.

L = Wave length (feet)

T = Wave period (seconds)

This results in,

$\xi_p = \frac{2.26\,T\,(tan\alpha)}{\sqrt{H_0}}$

The wave run-up R, is computed by the following equation:

R = $H_0(A\xi_p + C)\gamma_r\gamma_b\gamma_h\gamma_\beta$

Where,

R = Run-up on a relatively impermeable slope (i.e., the upstream slope of an embankment dam) (feet)

H_0 = Design wave height (feet)

ξ_p = Surf similarity factor

A, C = Coefficients dependent on ξ_p (See Table 3-6) and the probability of the run-up (2 percent is used for freeboard and riprap calculations)

$\gamma_r, \gamma_b, \gamma_h, \gamma_\beta$ = Reduction factors as under:

γ_r is a reduction factor to account for the roughness of the slope. To be taken from Table 3-5. For riprap a value of 0.55 may be taken.

γ_b is a reduction factor for the influence of a berm. (γ_b = 1.0 for non-bermed profiles).

γ_h is a reduction factor for the influence of shallow-water conditions, where the wave height distribution deviates from the Rayleigh distribution (γ_h = 1.0 for Rayleigh distributed waves).

γ_β is a reduction factor to account for reduction in run-up due to the direction of the fetch relative to the dam axis. (See Figure 3-8)

Knowing the surf similarity factor for peak waves i.e. ξ_p , Table 3-6 is used to determine the variables A and C for use in the above equation for 2-percent run-up. (Average of the highest 2 percent of the run-ups, which is commonly used in CEM, as well as for freeboard and riprap analysis).

For run-up calculations on most embankment dam's, γ_b and γ_h are set equal to 1.0.

b. Concrete/Masonry dams

Take it as equal to $1\frac{1}{3}$ of the design wave height as per the existing IS code 6512.

12) Wind Setup

Wind setup in feet, S, may be computed as follows:

$S = VMPH^2 F/1400D$

Where,

VMPH = Design wind velocity over water (mile/hr.)

F = Wind fetch (miles)

D = Average depth of water (feet)

The value of D should be a reasonable approximation of the average depth along the fetch length, with more emphasis given to depths within a few miles of the location for which the setup is being computed. The direction of fetch is taken as that of the central radial used in computing fetch.

However the wind set up may be insignificant in case of deep reservoirs.

13) Determine freeboard as indicated in 2 and 3 above.

14) Where reservoir submergence contour maps are not available, a preliminary freeboard study may be carried out by using ARC-GIS as under:

- Download the Jaxa DEM of the dam from www.earthexplorer.usgs.gov by giving its latitude and longitude.
- Import this DEM in ARC-GIS software.
- Identify the dam location using its latitude and longitude.
- Create contour of desired reservoir contour (FRL in this case) using spatial analysis tool in ARC-GIS.

In such cases the average depth required for estimating wind set up can be approximately calculated by dividing the reservoir capacity by reservoir area at the reservoir elevation considered (FRL in this case) using the available Area-Capacity curves.

3.2.2 Minimum Freeboard

1) It is the freeboard above the maximum reservoir level (MWL) obtained for the design flood.

2) For Embankment dams the minimum freeboard will be the sum of wind set up and run up corresponding to MWL condition.

3) For Concrete/Masonry dams the minimum freeboard may be taken as the sum of wind set up and $1\frac{1}{3}$ times the wave height corresponding to MWL condition.

4) The minimum freeboard should not be less than 1.50 m for Embankment dams and not less than 1.00 m for Concrete/Masonry dams. When the design flood is not the same as PMF then it may be checked that the MWL obtained for PMF is not higher than the top of dam.

5) The wind velocity at MWL over land may taken as one half to two third of that adopted for FRL condition.

6) This may be taken as wind velocity over land with duration of 1 min as taken for the fastest mile in USBR Technical Memorandum no.2 (1981 edition).

7) Wind velocities over land for 1 hr. , 2 hr. and any other duration may be calculated using Figure 3-9 which has been taken from the book Advanced Dam Engineering by Robert. B. Jansen. It is also available in the USACE EM 1110-2-1100 (Part II) 1 Jun 06 (Change 2). These wind velocities over land may be converted to wind velocities over water using figure 3-6.

8) Draw a curve between the above wind velocities over water and wind duration pertaining to the MWL condition.

9) For development of maximum wave height, three factors namely fetch distance, wind duration and the wind velocity are important.

   As per Design Standards no.13 of the USBR the minimum duration t_{min} (in hours) can be calculated as under:

   $$t_{min} = 1.87\,(F^{0.67} / VMPH^{0.34})$$

   where,

   F = Fetch (miles)
   VMPH = Wind Velocity over water (mile/hr.)

   Using the above relationship another curve can be prepared between wind velocities over water and minimum duration for the fetch at MWL calculated as described under normal freeboard above but using the MWL contour on the reservoir map.

   This curve can be plotted over the curve prepared at 8) above. The intersection of the two curves gives the wind velocity over water for use in MWL condition and the corresponding duration (See Figure 3-10). This methodology has been suggested as the present IS 10635 and IS 6512 do not consider the wind duration aspect.

The probability approach for determining the wind velocity for minimum freeboard as recommended in USBR Design Standard no.13, Chapter 6: Freeboard is not being suggested as the wind persistence data which is required i.e. data regarding different wind velocities being exceeded for different number of consecutive hours is normally not available.

However if such data can be obtained from a reputed agency like IMD then the method given in the above stated reference of USBR can be gainfully used.

10) Calculate significant wave height for MWL condition using the same formula as given under normal freeboard.

11) Design Wave Height

a) Embankment dams

Calculate Design Height (H_0) as under for minimum freeboard purposes:

For protected embankments $H_0 = H_s$

For unprotected embankments $H_0 = 1.27 H_s$

b) Concrete/Masonry dams

Take $H_0 = H_s$

12) Compute wave length, wave period, wave run-up and wind set up using the same formulae and methodology as given under normal freeboard earlier.

13) Determine freeboard as indicated in 2 and 3 above.

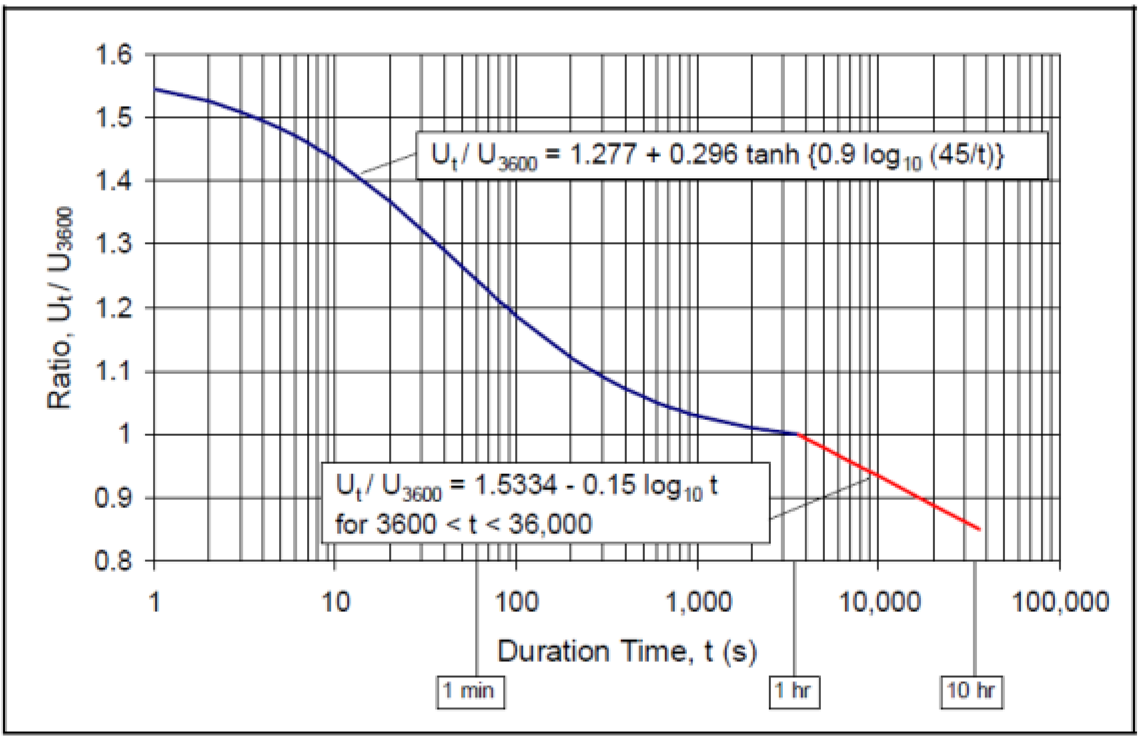


Figure 3-9 Ratio of wind speed of any duration U_t to the 1-hr wind speed U_{3600}

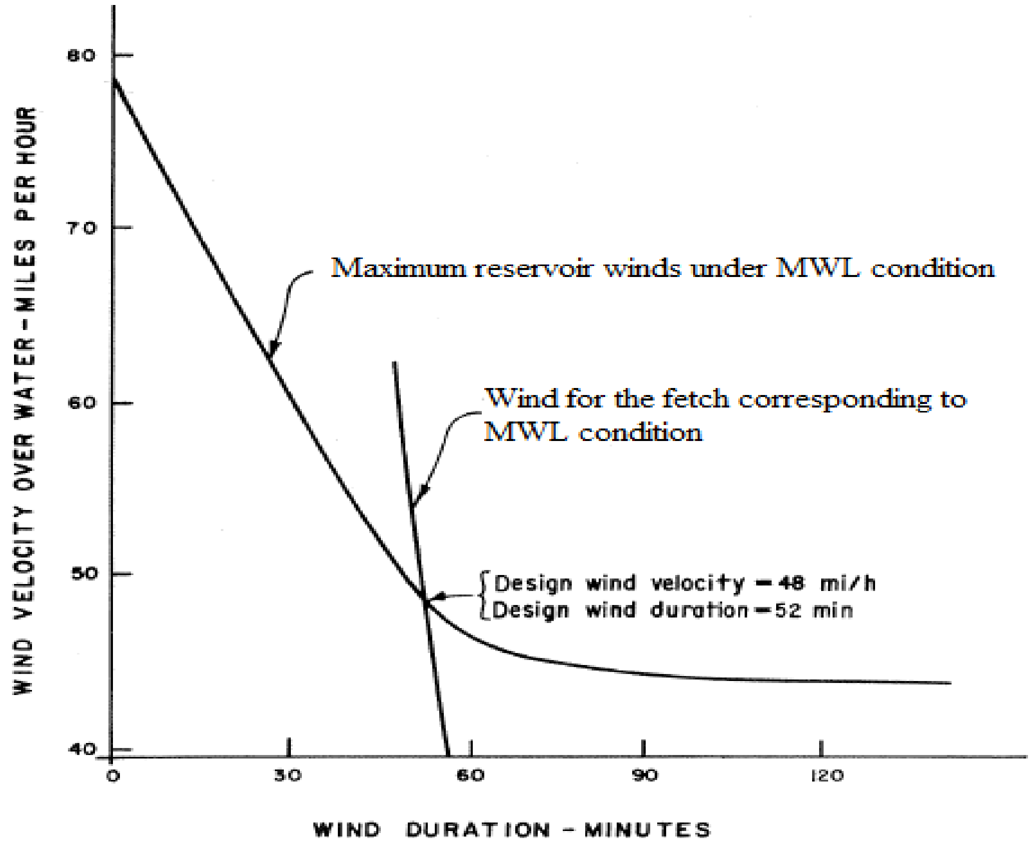


Figure 3-10 Plots of Wind Velocity over water and Wind Duration to determine the Design Wind Velocity and Design Wind Duration

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Chapter 4. INVESTIGATION

Investigations and testing to be carried out for a design review of an existing dam will vary from dam to dam and will depend on the issues involved and upon the studies to be conducted.

Investigations for an existing dam have also been covered in the Manual for Rehabilitating large dams. However a brief description of the same has been included in this Manual also as they provide the basic inputs required for reviewing the safety of any dam or for working out rehabilitation measures for any problem in the dam.

Following investigations/tests may need to be considered in an existing dam on a case to case basis depending on the requirements:

- Testing to determine various engineering parameters of the existing dam materials including non-destructive tests.
- Geo-Physical investigations in an existing dam (generally in concrete/masonry dams) to identify seepage zones/low density zones.
- Scanning of upstream face of concrete dams or any other component in a dam for mapping cracks, cavities etc. due to alkali-aggregate reaction/any other reason.
- Testing to determine various engineering parameters of the foundation rock/soil.
- Topographical survey of any particular area.
- Hydrographical survey to prepare revised area-capacity curve.
- Chemical analysis of seepage/reservoir water.

In case any additional spillway is required to be constructed then all investigations and testing as required for a new dam project may be required.

All investigation and exploration in the Himalayan Terrain are conducted as per guidelines framed by GSI (Geological Survey of India) detailed in Guidelines for investigations and explorations required at detailed project report (DPR) stage of proposed hydroelectric projects, in Himalayan Terrain and IS: 6955 & IS: 15662 for Earth & Rockfill Dams and Gravity Dams respectively.

4.1 Material Testing in Embankment Dams

The following is a list of field & laboratory tests that may be required to be conducted in respect of an existing embankment dam. Undisturbed samples will have to be taken out from the existing dam from desired locations & tested for various parameters. As regards additional materials that may be required for strengthening/ increase in dam section, suitable borrow areas will need to be identified. From these borrow areas disturbed samples can be taken for testing.

- Standard Penetration Test –IS: 2131
- Static Cone Penetration Test– IS: 4968 (Part - 3)
- Insitu Field Density Test
  - Determination of Dry Density of Soil In-Place by Core Cutter Method - IS: 2720 (Part-29)
  - Determination of Dry Density of Soils, In-Place, by Sand Replacement Method - IS: 2720 (Part-28)
  - Determination of Density of Soil In-Place by Rubber-Balloon Method - IS: 2720 (Part - 34)

- Determination of the Density In-place by Ring and Water Replacement Method - IS: 2720 (Part-33)
- Nuclear Density and Moisture Gauge Method for determination of in-situ density.
- Permeability of the existing embankment In-situ Permeability Test - IS: 5929.

- Undisturbed Soil Samples
  - Mechanical Analysis - IS:2720 (Part-4)
  - Atterberg limits - IS:2720 (Part-5)
  - Soil classification - IS: 1498
  - Insitu Density/Natural Moisture Content - IS: 2720 (Part -29)
  - Specific Gravity - IS:2720 (Part-3)
  - Triaxial Shear Test - Consolidated Undrained test with Pore pressure measurement - IS:2720 (Part-12)
  - Direct Shear Test IS:2720 (Part-13)
- Disturbed Soil Samples
  - Mechanical Analysis - IS:2720 (Part-4)
  - Atterberg limits - IS:2720 (Part-5)
  - Soil classification - IS 1498
  - Shrinkage Limit - IS:2720 (Part-6)
  - Standard Proctor Compaction - IS:2720 (Part-7)
  - Specific Gravity - IS:2720 (Part-3)
  - Triaxial Shear Test - Consolidated Undrained test with Pore pressure measurement - IS:2720 (Part-12)
  - One Dimensional Consolidation - IS:2720 (Part-15)
  - Laboratory Permeability - IS:2720 (Part-17)
- Expansive Soils
  - Differential Free Swell Index Test - IS:2720 (Part-40)
  - Shrinkage Limit Test - IS:2720 (Part-6)
  - Swelling Pressure Test - IS:2720 (Part-41)
- Dispersive Soils
  - Soil Dispersivity Identification Tests (standard procedures)
  - Pin-Hole Test - ASTM D4647
  - Double Hydrometer Test - ASTM D4221
  - Crumb Test - ASTM D6572
  - Chemical Analysis of Pore-Water Extract Test - ASTM D4542
- Chemical Analysis of Soil
  - pH - IS:2720-26
  - Total Soluble Salts - IS:2720 (Part-21)
  - Calcium Carbonate - IS:2720 (Part-23)
  - Water Soluble Sulphate - IS:2720 (Part-27)
  - Organic Matter - IS:2720 (Part-22)

4.2 Material Testing in Concrete/Masonry Dams

The structural integrity of a concrete dam depends on the strength of the concrete and foundation. Identifying Potential Failure

Modes (PFM) should guide the need to obtain various material properties. Typical PFMs are sliding and overturning of the dam. Sliding can occur within the dam body along lift joints or along cracks, along the dam to foundation interface, and along discontinuities in the foundation. The first task for assessing the need to obtain material properties in a concrete dam or foundation is to study the construction methods of the dam and results from previous field investigations and laboratory testing programs. A review of construction methods and photographs can help determine the lift joints strength and the roughness of the dam to foundation contact. Lift joint strength is a function of the concrete strength but also very importantly the construction technique. Lift joints have to be cleaned before subsequent concrete placements to obtain good bond. Construction photographs can reveal the care used to prepare the lift joints. However, the only true method to determine lift joint strength (bond) is by carefully coring vertically through all the lift joints. A concrete dam can have many seeps on the downstream face and have good lift joint bond while a dam with a dry downstream face can be totally unbonded. Sliding along the dam to rock interface is a function of the friction between the concrete and rock, but more importantly the roughness and asperities of the interface. The sliding resistance is greatly increased if this interface is very rough and undulating. As such, the bond between the concrete and foundation is less important then the physical nature of the interface. Optimally concrete core is extracted from the entire height of the dam at multiple locations with the dam with a minimum of 150-mm- (6-inch-) diameter core. Laboratory tests on the concrete should include density, compressive strength, Poisson's ratio, modulus of elasticity, splitting and direct tension of the parent concrete, direct tension on the lift joints, and direct shear tests of the parent concrete and lift joints. The concrete to rock interface should include direct shear tests (bonded and unbonded specimens). The bond of this interface is not as critical as joints in the foundation because joints immediately below the interface are assumed to not have any tensile strength or have little cohesive strength.

In general, the following tests may be required to be conducted.

- Non Destructive testing of concrete/ masonry – IS 13311 (Part-I)
- Density of Concrete/Masonry-IS 516
- Compressive strength of concrete/ Mortar in masonry - IS 516
- Static Modulus of Elasticity of concrete/ masonry – IS 516
- Water Permeability of Concrete / Masonry - IS 11216/DIN 1048
- Chloride Content & pH of concrete – IS 456
- Corrosion activity in concrete
- Water Quality Analysis of the reservoir water / seepage water – IS 3025
- Splitting tensile strength of parent concrete- IS 5816
- Direct tensile strength of concrete lift joints and parent concrete- ASTM D-2936
- Direct shear strength of lift joints, parent concrete, and foundation joints-ASTM D-3080
- Petrographic examination
- Ambient vibration testing or eccentric mass shaking of the concrete dam is being used to determine the natural frequencies of the dam and help calibrate the concrete and foundation properties used in the finite element models. First, finite element models are run with the laboratory tested material values and eigenvalue results compared

with measures natural frequencies. Then, laboratory values are adjusted until the natural frequencies match. This helps calibrate the dam and foundation modulus values.

4.3 Geophysical Investigations for masonry/ concrete dams for determination of permeable locations/zones

IS:15681 Code of Practice for geological exploration by Geophysical Methods (Seismic Refraction Method) deals with various aspects of seismic refraction technique and its application to shallow subsurface exploration of engineering sites. The primary purpose of the standard is to provide working knowledge of the method, with relevant references, and with a basis to weigh the applicability of the method to various engineering geological problems. In particular, it seeks to provide an understanding of the proper planning of surveys, so as to obtain adequate and relevant coverage and highlight the most important area of interpretation of seismic data.

In concrete/masonry dams with excessive seepage/leakage, sonic tomography can be considered for determing permeable zones/cracks/voids/cavities etc. This information can be helpful for finalizing the grouting pattern/details for control of seepage.

However, sonic tomography may not be effective as results are difficult to interpret and tests may not penetrate very deep into the concrete.

The sonic methods envisage generation of elastic energy (P waves) using various sources which is propagated through the investigated structure.
The elastic waves are recorded by specific sensor(Accelerometer) in the form of electric signals.

For transverse dam sections, geophysical surveys are performed following sonic tomography methods with transmission points located on the upstream side (reservoir side) while the receiver point are placed on the downstream side.

To generate P waves a "sparker" source is used, that produces explosive energy from a spark pulse generated between two electrodes in salt water. The compressional wave is transmitted through the water and therefore to the investigated surface (u/s face of dam). This sonic signal is recorded by the transducers (accelerometer) on the downstream side and transformed into electrical signals which is sent to recording unit through the connection cable.

Velocity analysis is then used for estimation of time needed by the elastic impulse to cover the distance between the transmitter and receiver .

Therefore, the second step consists in time-distance processing of data set to calculate sonic velocity distributions and to estimate cavities/low velocity zones and elastic properties of investigated area.

The results of the processing are plotted as colored tomograms, which show the variations of the P-waves velocity field, along with the representation of the measuring paths as obtained from the ray-tracing processing.

Typical Low Velocity & Low-Density Zones with Sonic Tomography in a dam block are shown in Figure 4-1 & 4-2.

Cross hole and Downhole technique may also be used to determine the low velocity zones and also the dynamic properties of materials. Indian Standard IS 13372 (Part 1) and (Part 2) describe the Seismic testing within a borehole and between the borehole respectively.

Another alternative is Ground Penetrating Radar.(GPR) survey which is conducted to identify shallow cracks, cavities and voids in the dam body based on contrast in Dielectric Constant. The method can be used to obtain high-resolution subsurface images showing cavities besides buried objects, cables pipes etc. The data of GPR is critical for interpreting l (SP) data and removal of voltage peaks generated due to buried metal objects, reinforcement etc.

4.4 Material testing of foundation rock/soil

Sometimes it is necessary to carry out foundation rock testing in case of concrete/masonry dams and foundation soil testing in case of embankment dams for reviewing their stability or for any other reason.

For foundation soil various tests on undisturbed samples that may be required to be carried out are discussed in 4.1 above.

Testing of the rock mass should include density, compressive strength, Poisson's ratio, RQD, and modulus of elasticity. Testing of critical discontinuities in the foundation should include direct shear tests and an assessment of the tightness and infilling of the joints. The deformation modulus of the foundation is determine by empirical methods using the compressive strength of the rock, RQD, joint orientation, etc.

Most failures of concrete dams worldwide have occurred in the foundation. The most critical assessment of the rock foundation is the orientation of discontinuities and the possibility of the formation of potentially removable blocks (rock wedges). It is critical that joints in the foundation under a concrete dam be mapped and characterized. If foundation mapping has not been done at a dam site, it should be performed.

A list of tests that may be necessary for foundation rock along with their relevant IS codes are given below.

- In Situ Tests
  - Uniaxial jacking test for deformation modulus of rock - IS 7317
  - In-situ shear test on rock-IS 7746
  - Pull-out test on anchor bars and rock bolts – IS 11309
  - In-situ determination of rock mass deformability using a flexible dilatometer-IS 12955 (Part 1 and Part 2)
  - Seismic testing of rock mass- IS 13372 (Part 1) within a borehole: Part 2) Between the borehole
  - Determination of rock stress- IS 13946 (Part 1 to Part 4)
- Laboratory Tests
  - Determination of Point load strength index of rocks-IS 8764
  - Determination of unconfined compressive strength of rock materials-IS 9143
  - Determination of modulus of elasticity and Poisson's ratio of rock materials in uniaxial compression-IS 9221
  - Determination of slake durability index of rocks-IS 10050
  - Determination of tensile strength by indirect tests on rock specimens-IS 10082
  - Determination of dynamic modulus of rock core specimens-IS 10782
  - Determination of direct shear strength of rock joints –IS 12634
  - Determination of water content, porosity, density and related properties of rock material –IS 13030
  - Determination of strength of rock materials in Tri-axial compression-IS 13047

- Methods for Laboratory Testing – Argillaceous Swelling Rocks-IS 14396 (Part 1 to 4)
- Determination of resistivity on rock specimens-IS 14436

4.5 Scanning of upstream face of concrete dams or any other component in a dam for mapping cracks, cavities etc.

The important concept to understand is that all concrete dams crack and that a crack is only critical if it leads to a Potential Failure Mode. For example Koyna Dam sustained significant cracking during the 1967 earthquake, but there was no uncontrolled release of the reservoir.

Scanning of u/s face of concrete dams or any other component in the dam is a very specialized work and is normally carried out by taking under water photographs/videos of the upstream face of the concrete dam or of any other affected component in a dam through experienced divers. These days' remote control vehicles are also being used for the purpose.

The information obtained can be used for studying the problems and working out remedial measures.

4.6 Determination of Site Specific Seismic Parameters

Where this study is required to be conducted, relevant literature will need to be referred to and a report is to be prepared.

The report is to be prepared in accordance with the Guidelines for preparation & submission of site specific seismic study report of river valley projects prepared by CWC & put up to National Committee on Seismic Design Parameters (NCSDP) for approval and finalization of the seismic parameters for the dam.

India's institutions are lacking the requisite capability in respect of modern seismic hazard analysis for dam sites, wherein CWC has to play a very important role to address this challenge within the organisation as well inthe country in order to ensure the adequate capability in this area within a definite time frame.

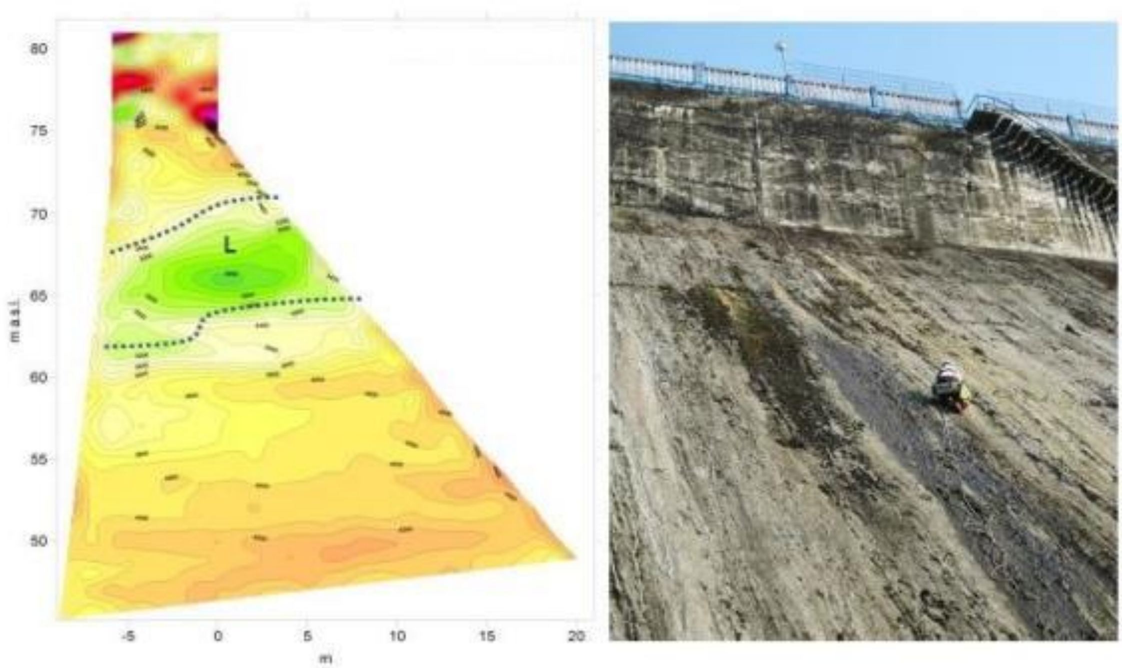

Figure 4-1: Typical Sonic Tomograms showing Velocity Contours and a view of the receiver points on downstream side

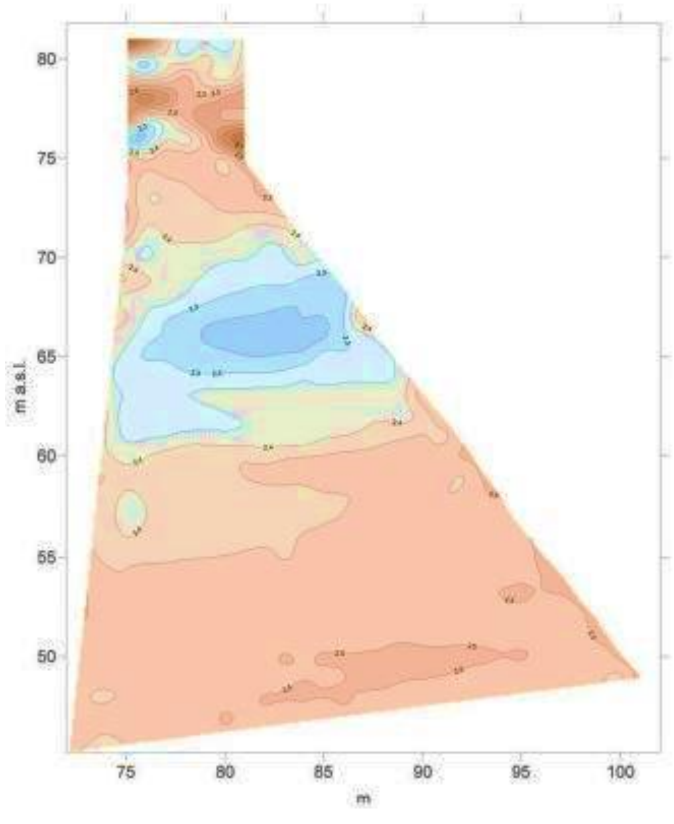

Figure 4-2: Typical Section showing Density Contours in Dam section

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Chapter 5. Surveillance And Performance Monitoring

Surveillance of dams during their construction, reservoir filling, and operation is an essential activity in a dam safety program. Surveillance plays a critical role in ensuring safety of existing dams, whose failure can result in unacceptable loss of life and economic losses. It should be emphasized that any time the reservoir goes above its previous historical maximum, the surveillance program should reinitiate the first filling protocol.

A robust surveillance process is the Owner's 'front line of defense' for the safe operation of their dams and reservoirs. Surveillance provides the cornerstone for effective management of dam safety and operational risks and includes visual inspections, instrument monitoring (including deformation surveys), data review and evaluation, and reporting on the safety of the dam.

The United States Federal Guidelines for Dam Safety (FEMA 2004) includes the following statement:

"Monitoring existing dams and reacting quickly to inadequate performance or to danger signals is a continuing critical aspect for dam safety. Careful monitoring and quick response can prevent failures, including those caused by poor construction."

Detailed description on project inspections is available in the Guideline for Safety Inspection of dams (Doc No. CDSO_GUD_DS_07_v1.0), CWC 2018. However an overview of the various types of inspections is given below:

5.1 Types of Inspections

Four different types of dam safety inspections are carried out for all dams:

1. Informal inspections
2. Scheduled inspections (Pre & Post monsoon inspections & other scheduled inspections)
3. Special (unscheduled) inspections
4. Comprehensive evaluation inspections

5.1.1 Informal Inspections

An informal inspection is a continuing effort by on-site personnel (dam owners/operators and maintenance personnel) performed during their normal duties. Informal inspections envisage surveillance of the dam periodically and are critical to the proper operation and maintenance of the dam. They consist of frequent observations of the general appearance and functioning of the dam and appurtenant structures.

Normally the dam owners, operators, maintenance crews, or other staff who are posted at dam site will make informal inspections. These people are the "first line of defense" in assuring safe dam conditions, and it is their responsibility to be familiar with all aspects of the dam. Their vigilance in inspection/surveillance of the dam, checking the operating equipment, and noting changes in conditions may prevent serious mishaps or even dam failures.

Informal inspections are important and should be performed at every available opportunity. These inspections may only cover one or two dam components as the occasion presents itself, or they may cover the entire dam and its appurtenant structures. The informal inspections are not as detailed as comprehensive evaluation, scheduled, and special inspections and will only require that a formal report is submitted to the dam owner's project files if a condition is detected that might endanger the dam.

5.1.2 Scheduled Inspections

Scheduled inspections shall consist of Pre-monsoon & Post-monsoon inspection and any other inspections carried out by the State Dam Safety Organization/any Expert panels constituted by the dam owner.

These inspections are performed to gather information on the current condition of the dam and its appurtenant works. This information is then used to establish needed repairs and repair schedules, and to assess the safety and operational adequacy of the dam. Scheduled inspections are also performed to evaluate previous repairs.

The purpose of scheduled inspections is to keep the dam and its appurtenant structures in good operating condition and to maintain a safe structure. As such, these inspections and timely maintenance will minimize the long-term costs and will extend the life of the dam. Scheduled inspections are performed more frequently than comprehensive evaluation inspections to detect at an early stage any developments that may be detrimental to the dam. These inspections involve assessing operational capability as well as structural stability and detection of any problems and to correct them before the conditions worsen. The field examinations should be made by the personnel assigned responsibility for monitoring the safety of the dam. If the dam or appurtenant works have instrumentation, the individual responsible for monitoring should analyze measurements as they are received and include an evaluation of that data. Dam Inspection Report or an inspection brief should be prepared following the field visit (Dam Inspection Report is recommended).

Scheduled inspections should include the following four components as a minimum:

1. Review of past inspection reports, monitoring data, photographs, maintenance records, or other pertinent data as may be required;
2. Visual inspection of the dam and its appurtenant works;
3. Preparation of a report or inspection brief, with relevant documentation and photographs.
4. Education and training if someone other than the owner is performing the inspection.

5.1.3 Special (Unscheduled) Inspections

Special inspections may need to be performed to resolve specific concerns or conditions at the site on an unscheduled basis. Special inspections are not regularly scheduled activities, but are usually made before or immediately after the dam or appurtenant works have been subjected to unusual events or conditions, such as an unusually high flood or a significant earthquake.

Japan Water Agency (JWA) has developed an excellent system of carrying out inspections after an earthquake event. For details refer "Inspection Manual for Dam Field Engineers after Seismic Events, Ichari Dam, Uttarakhand (CDSO_MAN_DS_01_v1.0), January 2018" The threshold acceleration adopted by them for carrying out inspection is when the acceleration recorded at dam foundation exceeds 25 gals (25 cm/sec^2). It is proposed to adopt their system in our guidelines. The system envisage a quick check within 1 hour after the earthquake event, next i.e. first check within 3 hours and a second check within 24 hrs.

The quick check will envisage:

- Confirming the seismic intensity at the dam site.
- Sending initial report regarding an assessment of a possible dam failure.
- Finding out urgent rescue needs.

The first inspection will envisage:

- Visual observations of leakage/ seepage, deformations, cracking in dam, slope failure, collapse of any component, functioning of gates and electrical devices, hill slopes (upstream and downstream of the dam), roads etc.

- Confirming any subsequent action to be taken.

The second inspection will envisage:

- Quantitatively measuring leakage, deformation and other monitoring items.
- Verifying function of facilities by actual movement.

Further the following activities are also recommended to minimize the adverse impacts of an earthquake

i) Regular field drills at dam site to make the site officials aware of their roles and responsibilities during and after an earthquake event and thereby to upgrade the earthquake response system

ii) Securing communication lines by having a redundancy in the system by way of availability of different types of telecommunication systems (viz. mobile phone, wireless, satellites, telephone etc.) at dam site.

iii) Securing adequate fuel for at least 3 days (viz. petrol, diesel) for the emergency power generators and other essential supplies like food, water, fire wood etc.

iv) Installation of seismometers in a dam and development of a data sharing system.

5.1.4 Comprehensive Evaluation Inspections

5.1.4.1 General

For comprehensive dam safety evaluation for each dam an independent panel of experts known as Dam Safety Review Panel (DSRP) needs to be constituted for determining the condition of the dam and appurtenant works. The panel would undertake evaluation of each dam once in 10 years or on occurrence of any extreme hydrological or seismic event or any unusual condition of the dam or in the reservoir rim. The terms of reference of the comprehensive dam safety evaluation shall include but not be limited to;

1. General assessment of hydrologic and hydraulic conditions, review of design flood, flood routing for revised design flood and mitigation measures.
2. Review and analysis of available data of dam design including seismic safety, construction, operation maintenance and performance of dam structure and appurtenant works.
3. Evaluation of procedures for operation, maintenance and inspection of dam and to suggest improvements / modifications.
4. Evaluation of any possible hazardous threat to the dam structure such as dam abutment slope stability failure or slope failures along the reservoir periphery.

5.1.4.2 Details to be provided to DSRP before inspection

All relevant details/data/drawings for the dam project to be inspected by the Panel of Experts shall be provided at least 3 months in advance of the proposed visit. This will include:-

(a) General Information

1. Scope of project
2. Basic data and salient features
3. Issues related to safety of dam
4. Details of key personnel
5. Emergency preparedness – Communications, Auxiliary Power, Downstream Warning system & Security of site.

(b) Hydrology

1. Description of drainage basin
2. Original inflow design flood
3. Spillway capacity at FRL & original MWL
4. Surface area & storage capacity of the reservoir
5. Flood routing criteria & results

(c) Geology

1. Dam site geology including geological reports
2. Quality and sufficiency of the geological investigations.
3. Special problems and their treatment
4. Reservoir competency as per geological report.
5. Slope stability issues along reservoir rim.

(d) Layout including Drawings

1. Dam
2. Spillway
3. Junction between Embankment & Concrete/Masonry dams
4. River/Canal outlets
5. Instrumentation

(e) Dam and Spillway

1. Geology
2. Special problems
3. Foundation treatment including treatment of faults/shear zones/ weak zones, curtain/consolidation grouting, drainage provisions, any other special treatment, cutoff trench, diaphragm walls etc.
4. Design criteria and result of stability analysis
5. Special studies (Finite element/Dynamic Analysis etc.)
6. Adequacy of design – from dam safety considerations
7. Hydraulic design of Spillway and Energy Dissipation Arrangements including past model study reports.
8. Instrumentation – analysis and interpretation of instrumentation data including structural behavior reports.
9. Pre-construction material testing reports including adequacy of field and laboratory investigations, appropriateness of materials selected etc.
10. Post-construction testing reports, if any.
11. Seismicity (Seismic Parameters approved by the National Committee for Recommending Seismic Design Parameters for Dams)

f) Construction history

g) Dam incidents/failures, remedial measures /modifications undertaken

h) Reservoir Operation & Regulation Plan

1. General
2. Reservoir filling
3. Water releases – normal and during floods.

5.1.4.3 Field Inspection – Observations & Recommendations regarding Remedial Measures

Each component of the project is to be inspected, evaluated and specific problems are to be brought out. Recommendations for necessary remedial measures need to be included in the panel's report. Various project components to be inspected shall include but will not be limited to;

(a) Dam

1. Upstream face
2. Downstream face
3. Top of dam
4. Structural behavior as observed visually and as per evaluation of

instrumentation data (any visible cracking, deflections etc.)

5. Seepage assessment
6. Condition of natural/excavated slopes in the abutments, both on u/s and d/s of the dam.
7. Any specific problems/ deficiencies

(b) Spillway

1. Civil structure
2. Energy Dissipation Arrangements (EDA)
3. Spill channel, drop structures etc. if any.
4. Condition of EDA and its performance
5. Spillway Gates & Hoists
6. Downstream safe carrying capacity of river / channel.

(c) River / Canal Outlets

1. Civil structures
2. Outlet Gates, Hoists & Controls
3. Conduits / Outlets through Embankment dams and sluices through Masonry / Concrete dams (Condition, problems etc.)
4. Trash racks, if any
5. Separate energy dissipation arrangements, if any.

(d) Review of Sedimentation of the Reservoir.

Assessment of sedimentation and its effect on flood routing, operation/ life of reservoir.

(e) Flood Hydrology

1. Extent & sufficiency of data available
2. Method used for estimating the design flood.
3. Design flood review study.
4. Flood routing studies with the revised flood
5. Adequacy of free board available

(f) Miscellaneous services /facilities

1. Access Roads / Bridges / Culverts
2. Elevators
3. Stand by power arrangements
4. Flood forecasting arrangements, if any
5. Communication facilities (Telephone, Satellite, Wireless, Mobile etc.)

(g) Hydraulic Model studies, if any new studies carried out.

(h) Earlier reports of experts / DSRP etc., if any, as annexures.

(i) Photographs of dam project showing problem areas.

5.1.4.4 Components involved

A comprehensive evaluation inspection of a dam will typically consist of five components:

1. Project records review (i.e. study of all design / construction records/ drawings, history of the dam's performance, past inspection notes/reports, notes on distress observed/ any rehabilitation measures undertaken earlier etc.).
2. Visual inspection or field examination of the dam and its appurtenant works.
3. Preparation of a detailed report of the inspection.
4. Education and training of the dam owner on the issues observed during dam inspection, identification of potential dam failure modes & to carry out additional field investigations & laboratory testing as required. Dam owners should be made part of the inspection process so that they take ownership of the results and are

committed to implementing the recommended remedial measures.

5. Design studies e.g. review of design flood, checking of the adequacy of spillway capacity, freeboard requirements, dam stability, any special study as required & submission of the report.
6. A comprehensive evaluation inspection should include a Potential Failure Modes Analysis workshop. After the team has reviewed the available material and performed their site visit, a PFMA workshop should be convened. The workshop should have a facilitator and the participants should be composed of a diverse group in the fields of structural, geotechnical, geology, materials, hydrology, hydraulics, mechanical, operations, instrumentation, and seismology. There is significant available literature on how to conduct a PFMA workshop (CWC-Risk 2019). Actually, a PFMA workshop is actually one of the more important first steps to take to determine the safety of a dam. The PFMA will help identify the critical findings of the Comprehensive Review and will prioritize the failure modes.

5.2 Performance Monitoring

5.2.1 Instrumentation

The Guidelines for Instrumentation of large dams (Doc. No. CDSO_GUD_DS_02 V1.0 January 2018) prepared under DRIP can be referred to for a detailed description. However a brief overview is provided here.

Various options and layouts are considered while planning instrumentation in dams and for monitoring their performance. Where possible, when determining what instruments are required to monitor the performance of a dam throughout its operational lifetime, Owners and Technical Advisers should adopt a 'simple and targeted' instrumentation philosophy.

All dam instrumentation should have a clear purpose that is linked to one or all of the following objectives:

- Improving the understanding of a dam or foundation's characteristic behavior during normal operation, and during unusual and extreme events.
- Providing early indication of the onset of potential failure modes for a dam.

Instrumentation can assist with the identification of the trends or conditions that are indicative of a potential failure mode. For example, uplift measurements in gravity dams can give an idea whether the uplift pressure are more or less than the design values. It can also provide an idea regarding the condition of foundation drainage holes viz. whether in working condition or choked. If choked then cleaning / re-drilling of holes will be necessary from dam stability considerations.

Dam performance monitoring instruments should be robust, durable, require little maintenance and able to be read easily and consistently, often by non-specialist personnel. That is, it should measure as directly as possible a parameter, condition or quantity that supports the aforementioned dam performance monitoring objectives. The operational lifetime of a dam is typically tens of decades, and the surveillance instrumentation should be selected so that either it has a similar lifespan, or that components with a shorter life can be safely maintained and/or replaced. The instrumentation data should be graphed and reviewed in a timely manner by a qualified engineer. Threshold and Action Levels should be established on the graphs and the appropriate responses to be taken when these levels are exceeded. Also, the instrumentation should be linked to a Potential Failure Mode.

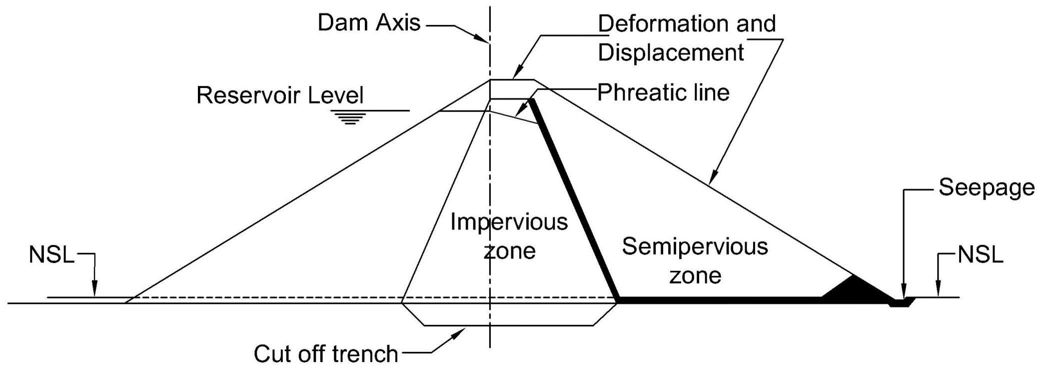

Figure 5-1: Embankment Dam Performance Parameters

The overall dam instrument layout / array should be resilient and should provide for redundancy as appropriate.

Redundancy is specifically important for dams where piezometric (or uplift) information is measured using vibrating wire instruments, or where it is gathered and reported using telemetry or other means of electronic transmittal that can be affected by lightning strikes or power loss. In such cases backup manual measurements of embankment piezometers or uplift pressures in concrete dams at key locations should be provided.

Survey monuments installed to allow measurement of a dam's deformation or settlement (or the displacement of an appurtenant structures) are not typically considered to be 'instrumentation'; however, they do provide the same function in that they can yield important information relative to some potential failure modes and allow the behavior of the dam to be monitored.

Dam performance monitoring instruments predominantly measure geotechnical, hydrologic or structural parameters.

The need for and value of dam performance monitoring instrumentation will depend on the requirements for the particular dam. Most instrumentation is selected during dam design and installed during construction, and may have a primary purpose related to the monitoring of construction-related parameters rather than those parameters required for the long- term management of dam safety. Hence, it may be appropriate to consider additional instruments to ensure dam performance monitoring needs are met or, where instruments are found to be redundant, it may be appropriate to decommission instruments. Additional or different instrumentation may also be installed when a potential dam safety deficiency is being investigated and assessed.

Technological advances in instrumentation types and systems will occur over the life of any dam. It is therefore likely that the original instrumentation will be augmented or replaced by new systems over time. Where possible, a period of monitoring overlap should occur to ensure that historical data can be correlated to information obtained from new systems.

5.2.2 Key Dam Performance Parameters and Instrument Types

Universal to all dams, the most important parameters that need to be measured quantitatively and evaluated are:

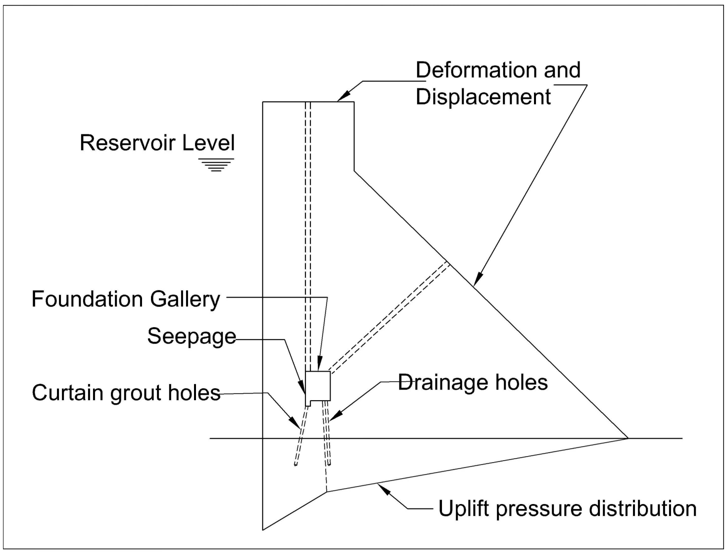

Figure 5-2 Concrete Dam Performance Parameters

- Reservoir and tailwater levels.
- Reservoir inflow and outflow levels
- Accelerometer in high seismic areas.
- Dam and foundation seepage and/or leakage rates.
- Dam/abutment internal water pressures and phreatic surfaces.
- Foundation uplift pressures.
- Dam deformation and displacement.

The above key parameters for embankment and concrete / masonry dams are shown diagrammatically in Figures 5.1 and 5.2 and are discussed in the sections that follow.

5.2.2.1 Reservoir Level

The Reservoir level is a fundamentally important measure of the loading on the dam and therefore the head that the dam and its foundation are subject to and the freeboard available to avoid overtopping. As a minimum, reservoir level should be recorded whenever visual inspections and instrumented measurements are carried out so that the effect of the reservoir loading with reservoir at different levels on the various engineering parameters can be studied /evaluated.

While water level sensor instruments are commonly employed (allowing automated and frequent monitoring), a water level staff gauge that can be read manually should be installed in all reservoirs. Water level staff gauges are simple, effective and reliable (they do not need a power source or have any electronic components) and where water level sensors are installed they provide an important calibration check.

Water level staff gauges should be dimensioned to allow measurement of the fully operational (including flood) range of reservoir levels and positioned so that they can easily be read in all loading and weather conditions. They should also be sited to allow reading without placing personnel at risk. Reservoir level should optimally be measured in metres above mean sea level for ease of correlation with dam features and other measured performance parameters such as piezometric levels and foundation uplift, seepage etc.

5.2.2.2 Seepage and/or Leakage Rate

Seepage and/or leakage rate in an embankment dam is an indicator of the performance of impermeable (or low permeability) elements installed in the dam and foundation, and the performance of the abutments and foundation where no impermeable elements are installed. The objective of measuring seepage flows is generally more about the identification of seepage trends and understanding the overall performance of the dam, rather than the recording of absolute values. Decreasing seepage flows may need to be scrutinized just as much as increasing seepage flows as they may indicate a change that is unacceptable.

The ability to measure rate of seepage and leakage through the embankment dam, its foundation or abutment usually relies on directing the seepage or leakage, through appropriate collection and drainage facilities, to a measurement point close to the dam's toe or at the location where the seepage or leakage emerges from the dam, foundation or abutment.

Seepage and leakage flow is best measured volumetrically, either by measuring the time to fill a container of known volume, or by installing a weir or flume with a theoretical (or calibrated) rating that allows the measured head to be converted to flow rate. For the purpose of ongoing monitoring and evaluation of a embankment dam's performance the most important aspect of seepage and leakage rate measurement is repeatability, rather than absolute precision. Weirs should be sized for the anticipated flows and weir boxes should be large enough to provide calm water surfaces behind the weir plates. In some cases baffles may be needed to achieve this. V-notch weirs provide precision for the measurement of seepage flows; however, for large flows, broad crested weirs or flumes will be necessary.

The observation of seepage and leakage flows via the use of weir also allows the detection of any materials being transported by the seepage flows. The detection of turbid seepage or soil particles in seepage flows is important as they may be an indicator that internal erosion (backward erosion or piping or washing of the fines) is taking place within the dam, in its abutments or in the foundation. In order to detect whether or not soil particles in a weir are the results of internal erosion, the weir may have to be covered to protect it from windborne material and periodically cleaned to enable the captured material to be examined and weighed.

In case of Masonry/Concrete dams the seepage is measured in foundation gallery as well as in inspection galleries at higher levels. Excessive seepage is an indicator of poor quality of work, existence of low density areas, voids / segregation, poor lift joints in concrete dams etc. Upstream pointing, grouting of dam body in masonry dams & grouting of lift joints in concrete dams may be required is such cases.

Monitoring of any erosion or transport of material is important. As with Camera Dam in Brazil, joint material in the foundation was eroding away causing a piping failure under the concrete dam.

5.2.2.3 Internal Water Pressure and Foundation Uplift Pressure

Internal water pressure and foundation uplift pressure are measured to allow the stability of the dam to be evaluated and to compare them with design assumptions. The absolute measured values are therefore of prime importance; however, changes recorded over time also need to be examined and understood. Water pressure is usually measured using a piezometer. Internal piezo-metric pressures are most relevant to embankment dams as well as their foundations and abutments.. The measurement of internal water pressure at a number of points in the body of the embankment dam, or in its abutments or foundation, allows the phreatic surface (below which the materials are saturated) to be understood. Saturation of the downstream shoulder of an embankment dam is undesirable for dam stability.

Also Uplift pressure at or near the toe of embankment dams may also be relevant if a blowout condition or potential piping condition exists.

Uplift pressures are .also most relevant to concrete/masonry dams and their foundations, and allow their stability to be evaluated.

There are a number of piezometer types including Open Standpipes (Observation wells), Hydraulic, Pneumatic, Vibrating wire and Fiber Optic piezometers. Piezometers are typically installed during the construction of a dam in its body or foundation. This makes the replacement of certain types of piezometers difficult and potentially risky process. Therefore the maintenance of installed piezometers to preserve their accuracy and maximise their service lives, is very important and usually requires the input of an appropriately skilled and competent Technical Adviser (specifically a geotechnical Instrumentation specialist). Where such embedded piezometers malfunction, backup piezometers that are long lasting should be considered. For retrofitting or replacement of piezometers (e.g. for replacing failed instruments, characterization of a special feature or the monitoring of a potential failure mode), extreme care should be taken in planning and installing the instruments to avoid damage to the dam and its foundation. An appropriately experienced Technical Adviser or Technical Specialist should be consulted in such cases. In some cases the dam safety risks associated with installing a new piezometer may outweigh the benefits of the instrument.

Foundation drainage holes in concrete dams can be used for installing piezometers, either by measuring the depth to the water level (if the water level is below the top of the drain) or by installing a pressure gauge over the steel pipe at the top of foundation drainage holes (if water is flowing from the drain). An appropriately experienced Technical Adviser or Technical Specialist should be consulted in such cases. For correct evaluations of dam performance it is important that the locations of piezometers in the body of a dam or foundation are accurately known (position and level), that the instruments are correctly identified, that their precision and accuracy are regularly assessed, and that they are appropriately maintained.

5.2.2.4 Deformation and Displacement

Deformation and displacement in dams can be effective performance indicators for settlement, loss in freeboard and a number of other potential failure modes. They are also useful to characterize the behavior of dam and foundation components. They are most commonly observed by visual observation during routine surveillance, and measured by traditional survey methods such as precise levelling and Electronic Distance Measurement (EDM) of targets installed at key locations on the dam and its foundation. Visual observations can generally only identify large or obvious deformations or movements in a structure or abutment. Instrumented measurement and surveying are the most effective methods for measuring and monitoring changes at specific locations and features, and establishing movement trends or verifying visual inferences of movement. A Designer with experience in the particular dam type should be consulted when designing a dam deformation survey layout to ensure that the dam's performance monitoring objectives are met. In addition to traditional survey methods, there are a number of alternative methods and technologies available for the measurement of deformations and displacements. Examples include pendulums, inclinometers, tilt meters, joint meters, Global Navigation Satellite Systems (GNSS), continuous survey monitoring (CSM), robotic total station and laser scanning (ground mounted or airborne). Fundamentally, the method and/or technology adopted should be selected such that it meets the dam performance monitoring objectives related to precision and accuracy, and can be readily calibrated.

For accuracy in measurements deformation surveys should be conducted by specialist surveyors with equipment and methodologies that achieve the required precision and accuracy (within 1 to 2 mm vertically and 3 to 4 mm horizontally). A survey control network on stable ground remote from the dam structure should be utilized to minimize survey errors and a specialist surveyor should be consulted in designing the control network. Generally, the size of the structure and its survey control network will influence the achievable precision and accuracy of the deformation survey. To be reproducible and to detect changes, periodic surveys should generally be taken at the same time of year. Also, when surveying methods or survey personnel change, a close examination of the results should be carried out to establish the validity of the results and their correlation with past surveys.

Vegetation management plays a significant part in the effectiveness of deformation monitoring. For visual observation, clear dam and abutment faces allow the identification of surface anomalies. For instrumented surveys, vegetation and man-made additions (e.g. handrails or fences) may block lines of sight between survey pillars and monitoring points.

5.2.2.5 Other Instruments and Systems

There are a vast range of other instruments and systems also which are used for the monitoring of dam performance and the monitoring of hazards. Some common examples include, but are not limited to:

- The use of cement plaster across cracks in concrete dams on the crest or within galleries to monitor relative movements. Two or three dimensional crack monitoring devices can also be attached to the dam for greater accuracy. An easy crack monitor is to have inspectors at the site mark the end of a crack with a perpendicular line and date. This provides a visual log of the crack progression with time.
- Dye tests for determining seepage and leakage origins/paths.
- Turbidity meters (indicators of internal erosion).
- Video cameras for real-time visual observations, including the internal inspection of conduits (drains and outlet tunnels) both above and under water.
- Thermometers for recording temperature and temperature gradients in concrete dams (for thermal studies).
- Trip wire systems (e.g. displacement/rupture of an active fault, or a dam itself).
- Post-tensioned cable anchor load testing (to confirm anchor tensions).
- Temperature sensing systems for the identification of seepage in dams or foundations (e.g. distributed temperature sensing and resistance temperature devices). Temperature sensors can provide valuable data on the flow time and flow source of seepage water, particularly when complemented by other measured parameters such as piezometric pressure, seepage flow rate, and the temperature of the reservoir and other potential sources (such as ambient groundwater or tail water).
- Early warning upstream rainfall collection and catchment modelling systems for predicting the size of incoming floods or extreme weather conditions (an important aspect for surveillance and emergency preparedness).
- Rainfall measurement to assist with the interpretation of seepage observations, and the evaluation or correlation of landslide and abutment slope movements.
- A seismic monitoring network for detecting and notifying the location and strength of earthquakes (an important aspect for emergency response). The India Meteorological Department Seismic network is available
- Strong motion seismic sensors for the measurement of ground motions. These may be helpful where the IMD network coverage is limited and/or

where measurement of ground motions at the dam site is required. The locations for installation of strong motion recorders should be based on the site conditions and preferred locations. In order of usefulness the preferred location are:-

- the base of the dam to record the peak ground acceleration
- the abutments to record topographic amplification of the peak ground acceleration
- the dam crest to record the amplification of the peak ground acceleration.

Instruments and systems as indicated above may be built into or near a dam at the time of its construction or added during the life of a dam to supplement or enhance existing instrumented monitoring, to address a specific potential failure mode, or to investigate a potential or confirmed dam safety deficiency.

5.2.3 Various Parameters Measured and the Suggested Frequency of Measurements

Various parameters to be measured in dams & suggested frequency of readings for specified instruments as prescribed in other guidelines viz. Instrumentation for dam & O&M for dam are given at Tables 5-1 & 5-2 for reference**.** All instruments should be read immediately after seismic activity or historic reservoir levels.

Many of the instruments in Tables 5-1 & 5-2 should be read daily during initial filling or anytime the reservoir goes above the historic maximum; weekly is too infrequent. First filling is a critical timeframe for a dam. Also, the dam should have continual visual monitoring during initial filling or any time the reservoir goes above the historic maximum. A Potential Failure Mode workshop can help establish the need for certain instruments and the reading frequency. As a minimum, instrument readings at a concrete dam should include reservoir levels, dam deflections, seepage through the dam, and uplift pressures under the dam. It is suggested that some instruments be read weekly for the first year and some monthly to develop a detailed plot of data. Then the frequency of readings can be reduced as indicated in the Table. Preferably, instruments should be read at the same time of the dam and the same day of the month.

5.3 Dam Performance Evaluation

Experienced Engineers should be assigned the job of establishing performance expectations and to evaluate dam performance appropriate to the consequences of failure and the complexity of the dam being evaluated. In some situations, Technical Specialists may be required (e.g. complex foundation and/or dam behavior, complex structural or geotechnical analysis, high or extreme consequences of failure, or the management of a dam safety deficiency).

The evaluation of visual observations and instrumented data with respect to a dam's safe performance is an essential part of a dam safety programmer.

Performance evaluation should be undertaken following the completion of each routine surveillance inspection in a timeframe that reflects the dam condition and performance. Besides during normal operations of dam, this exercise needs to be carried out after unusual events like high flood or earthquake as also indicated in the Guidelines for Safety Inspection of Dams & Guidelines for preparing O&M Manuals for Dams.

The completion of an effective evaluation requires an understanding of the dam's behavior under all loading conditions, and the use of evaluation techniques that can predict the expected behavior of the dam, based on the available information of the dam (e.g. design, construction, operation and maintenance records, rehabilitation records, and records of unusual events and incidents) and then compare it with the actual surveillance records. Importantly, the evaluation must consider the dam's 'performance as a whole' in the context of the dam setting, design philosophy, construction features, condition, historical performance and potential failure modes. Judgments should not be made based solely on isolated observations or instrument readings. Instead, the wider dam and foundation context should be considered, with conclusions drawn and supported by bringing together a range of relevant performance parameters and other information relevant to the safety of the dam.

For this purpose structural behavior reports need to be prepared based on the instrument data collected. These reports can be examined by the designers.

While reviewing the safety of existing dams it is desirable to include the following aspects in the structural behavior reports:

- Comments on the actual structure behavior based on: an understanding of the dam's characteristic behavior – how the dam and its foundation should typically behave under various loading conditions, comparison of the actual parameters measured with the design assumptions/parameters.

- The potential failure modes of the dam, key performance indicators and condition of the dam. Potential failure modes are an extremely important concept for engineers, owners, and maintenance personnel.

- Established two alert thresholds (acceptable performance limits) for key performance indicators coupled with action items. For instance, a reservoir level that starts a preparedness level, beginning actions, and then possible evacuations. Other terms for this is the Ready, Set, Go levels. Also, every concrete dam should have "safe" water levels determined for the key reservoir levels that indicate the dam is within criteria, starts developing tension at the heel, and starts overturning.

Table 5-1: Parameters to be Monitored at Dams

Structure Type	Feature	Visual observation	Movements	Uplift and pore pressure	Water levels	Seepage flows	Water quality	Temperature measurement	Crack and joint measurement	Seismic measurement	Stress-strain measurement
Embankment Dams	Upstream slope	•	•	•	•	–	–	–	–	•	–
	Downstream slope	•	•	•	–	•	•	•	•	•	–
	Abutments	•	•	•	–	•	•	•	–	•	–
	Crest (Dam Top)	•	•	•	–	–	–	–	•	•	–
	Internal drainage system	–	–	•	–	•	•	•	–	–	–
	D/s Toe Drains Relief Drain	•	–	•	–	•	•	–	–	–	–
	U/s Riprap and D/s slope protection	•	–	–	–	–	–	–	–	–	–

Structure Type	Feature	Visual observation	Movements	Uplift and pore pressure	Water levels	Seepage flows	Water quality	Temperature measurement	Crack and joint measurement	Seismic measurement	Stress-strain measurement
Concrete and Masonry Dams	Upstream slope	•	•	–	•	–	–	•	•	•	•
	Downstream slope	•	•	•	–	–	–	•	•	•	•
	Abutments	•	•	•	–	•	•	–	–	•	•
	Crest (Dam Top)	•	•	•	–	–	–	•	•	•	•
	Internal drainage system in Dam Body	–	–	•	–	•	–	–	•	–	–
	Foundation drains	•	–	•	–	•	–	–	–	–	–
	Galleries	•	•	–	–	–	–	–	•	•	•
	Sluices / controls	•	–	–	•	–	–	–	–	–	–
Spillways	Approach channel	•	•	–	•	–	–	–	–	–	–
	Control structure	•	•	•	•	•	–	–	•	•	–
	Stilling basin / any other EDA	•	–	–	•	–	–	–	•	–	–
	Discharge conduit/channel	•	–	•	•	–	–	–	–	–	–
	Gate controls	•	–	–	–	–	–	–	–	–	–
	Erosion protection on d/s of EDA	•	–	–	–	–	–	–	–	–	–
	Side slopes	•	•	•	–	•	–	–	–	–	–
Outlets	Control Structure	•	•	•	•	–	–	–	•	•	–
	Stilling basin / any other EDA	•	–	–	–	–	–	–	–	–	–
	Discharge conduit/channel	•	•	•	•	–	–	–	•	–	–
	Trash rack/debris controls	•	–	–	–	–	–	–	–	–	–
General Areas	Reservoir surface	•	–	–	–	–	•	–	–	–	–
	Mechanical/ electrical systems	•	–	–	•	–		–	–	–	–
	Reservoir Periphery	•	–	–	–	–	•	–	–	–	–
	Upstream watershed	•	–	–	–	–	•	–	–	–	–
	Downstream channel	•	–	–	–	•	•	–	–	–	–
	Emergency Warning System	•	–	–	–	–	–	–	–	–	–

Table 5-2: Suggested frequency of readings for specified instruments

Type of instrument	During Construction		During initial filling	During Period of Operation		
	Construction	Shutdown		Year 1	Years 2 to 3	Regular
Vibrating wire piezometers	W	M	D	BiW	M	M
Hydrostatic uplift pressure pipes	W	M	D	W	BiW	M
Porous-tube piezometers	M	M	D	W	M	M
Slotted-pipe piezometers	M	M	D	W	M	M
Observation wells	W	M	D	W	BiW	M
Seepage measurement (weirs and flumes)	W	M	D	W	M	M
Visual seepage monitoring	W	W	D	W	F	M
Resistance thermometers	W	M	W	W	M	M
Thermocouples	D	M	W	W	M	M
Carlson strain meters	W	W	W	BiW	M	M
Joint meters	W	W	W	BiW	M	M
Stress meters	W	M	W	BiW	M	M
Reinforcement meters	W	M	M	M	M	M
Penstock meters	W	M	M	M	M	M
Deflectometers	W	M	W	W	M	M
Vibrating wire strain gauge	W	M	M	M	M	M
Vibrating-wire total pressure cell	W	M	M	M	M	M
Load cell	W	M	W	BiW	M	M
Pore pressure meters	W	W	D	W	M	M
Foundation deformation meters	W	W	W	BiW	M	M
Flat jacks	D	W	W	BiW	M	M
Tape gauges (tunnel)	W	W	W/BiW	BiW	M	M
Whitmore gauges, Avongard crack meter	W	M	W	W	M	M
Wire gauges	W	M	W/M	W/M	M	M/Q
Abutment defor-	W	M	W	W	M	M

Type of instrument	During Construction		During initial filling	During Period of Operation		
	Construction	Shutdown		Year 1	Years 2 to 3	Regular
mation gauges						
Ames dialmeters, differential buttress gauges	W	M	W	W	M	M
Plumblines	D	W	D	W	BiW	M
Inclinometer	W	W	W	W	BiW	M
Collimation	Every two days for a month	M	W	BiW	M	M
Embankment settlement points	--[c]	--	M	BiM	Q	SA
Level points	M	Q	M	M/Y	BM/Q	BM
Multipoint extensometers	W	M	W	M	M	Q/SA
Triangulation			M	M	Q	SA
Trilateration (EDM)	--	--	BiW/M	M	Q	Q/A
Reservoir slide monitoring systems	--	--	M	M	M	Q
Power plant movement	--	--	M/W	M	M	M/Q
Rock movement	W	M	W	M	M	M

1. These are suggested minimums. However, anomalies observed or unusual occurrences, such as earthquakes or floods, will require additional readings.
2. D = daily, W = weekly, BiW = bi-weekly, M = monthly, Q = quarterly, SA = semi-annually, A = annually.
3. Shutdown is that period during construction when the works remained suspended / stopped, due to any reason.

Chapter 6. CONCRETE AND MASONRY DAMS

6.1 Introduction

There are various types of concrete and masonry dams like gravity dams, arch dams, buttress dams, hollow gravity dams etc. Out of them the solid gravity dams are the most common and are the simplest type to design and build. In India almost all concrete and masonry dams constructed are Gravity dams.

Gravity dams resist the imposed forces by their self-weight alone. They are usually straight in plan. Sometimes a slight curvature is provided in plan to accommodate one or two extra spillway blocks/bays e.g. in Indira Sagar dam, Madhya Pradesh. At times the dam alignment is along two or more straight lines which are at an angle in plan as per site-specific conditions.

The assessment of safety of an existing dam often involves investigations and design studies which are similar to those needed for the design of new dams. In many instances, there is insufficient data, obsolete data, missing data, or inadequate data by today's standards with the project authorities to perform an adequate engineering review. Hence fresh geological, hydrological and / or seismological studies along with detailed investigations for determining the engineering properties of the concrete / masonry and foundation may be necessary.

The investigations may envisage taking out and testing core specimens from the dam structure and its foundation. Sometimes non-destructive tests such as rebound hammer or sonic velocity measurements are conducted which give qualitative results only but may be helpful in overall evaluation.

Review of stability of an existing dam may become necessary either because the original loading conditions and methods of analysis no longer conform to current design standards and practices or due to non-consideration of earthquake effects earlier or due to change in seismic zone or due to increased maximum water level on account of an upward revision in design flood etc. Also in the recent years new criteria have evolved for carrying out a more realistic dynamic analysis for dams.

6.2 Some Common Features of Gravity Dams

6.2.1 General

Historically most of the Gravity dams built in India were Masonry dams using lime-surkhi mortar in the earlier times and thereafter with cement mortar subsequently. Masonry dams were preferred to Concrete Gravity dams mainly because of shortage of cement (they consumed less cement compared to concrete gravity dams) and because they were labour intensive and provided a lot of employment facilities and also due to the availability of good skilled masons at that time.

Subsequently in the latter half of the last century a lot of seepage problems were reported in a large number of masonry dams built during that time. This was mainly due to poor workmanship and non-availability of good masons. This resulted in improvements like provision of concrete membrane on the upstream of many of the new Masonry dams and also pre-packed masonry construction.

However these days as cement is no longer a scarce commodity, Concrete Gravity dams are generally being preferred over Masonry dams.

Most of the Concrete dams built in India are Conventional Concrete Gravity dams along with a few Roller Compacted Concrete Gravity dams.

6.2.2 Dam Sections

Gravity dams mainly consist of non-overflow and overflow (spillway) sections.

Where a power house is to be located at the toe of the dam, some non-overflow blocks are converted into power dam blocks. These power dam sections contain intake structure with water passages, embedded penstocks, gates, air vents etc.

General sectional details of a typical Non-Overflow section, Spillway section and a Power Dam section of a Gravity dam are illustrated in Figures 6-1, 6-2 and 6-3.

The upstream and downstream slopes and the base width of dam sections are worked out from stability considerations.

The non-overflow sections usually have a uniform downstream slope that when projected intersects the upstream face near the maximum reservoir level/dam top level. To meet stability requirements the downstream face is provided a slope which is usually of the order of 0.70 - 0.85 horizontal to 1.0 vertical. The upstream face is normally vertical but sometimes an upstream batter may be required from stability considerations.

Both the upstream and downstream faces of gravity dams are normally provided with fillets at their intersection with the foundation to reduce stress concentrations.

Also abrupt changes of slope on either face of the dam can cause unacceptable stress concentrations and should be avoided whenever possible. At intersections between the vertical and sloping faces of the dam circular curves tangent to each face are desirable to reduce stress concentrations.

The crest (dam top) of the non-overflow section is usually dimensioned to provide for a roadway and the desired freeboard.

In areas of significant seismicity the mass of the crest should be kept to a minimum.

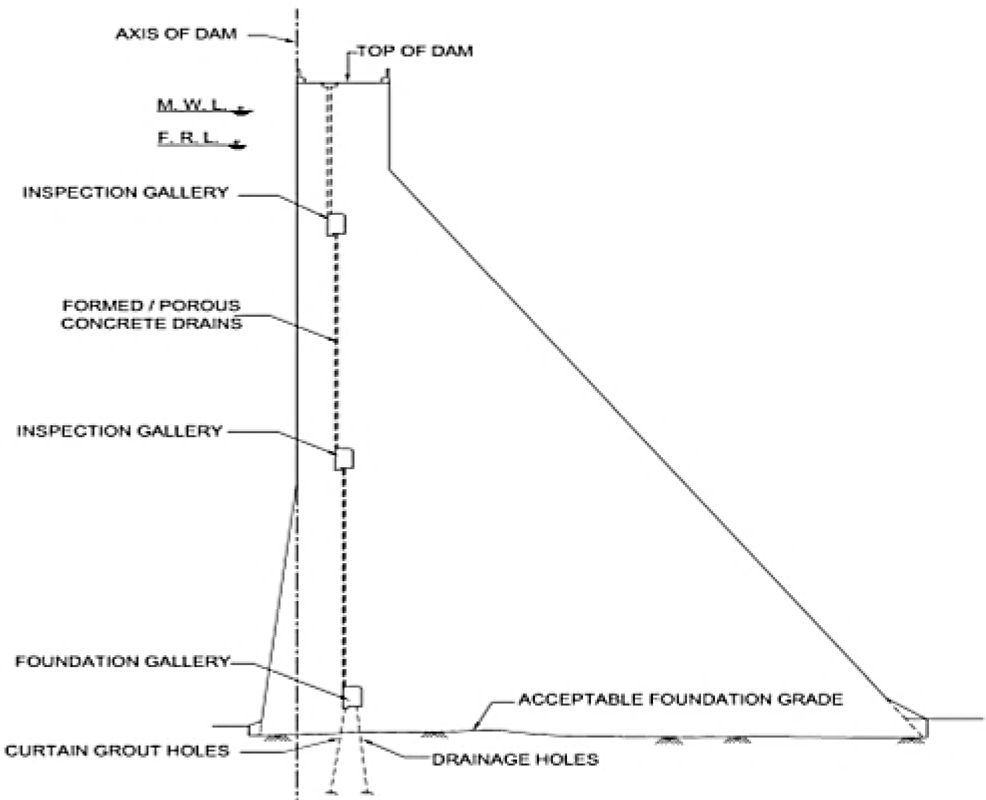

Figure 6-1: Typical Non-Overflow Section of a Gravity dam

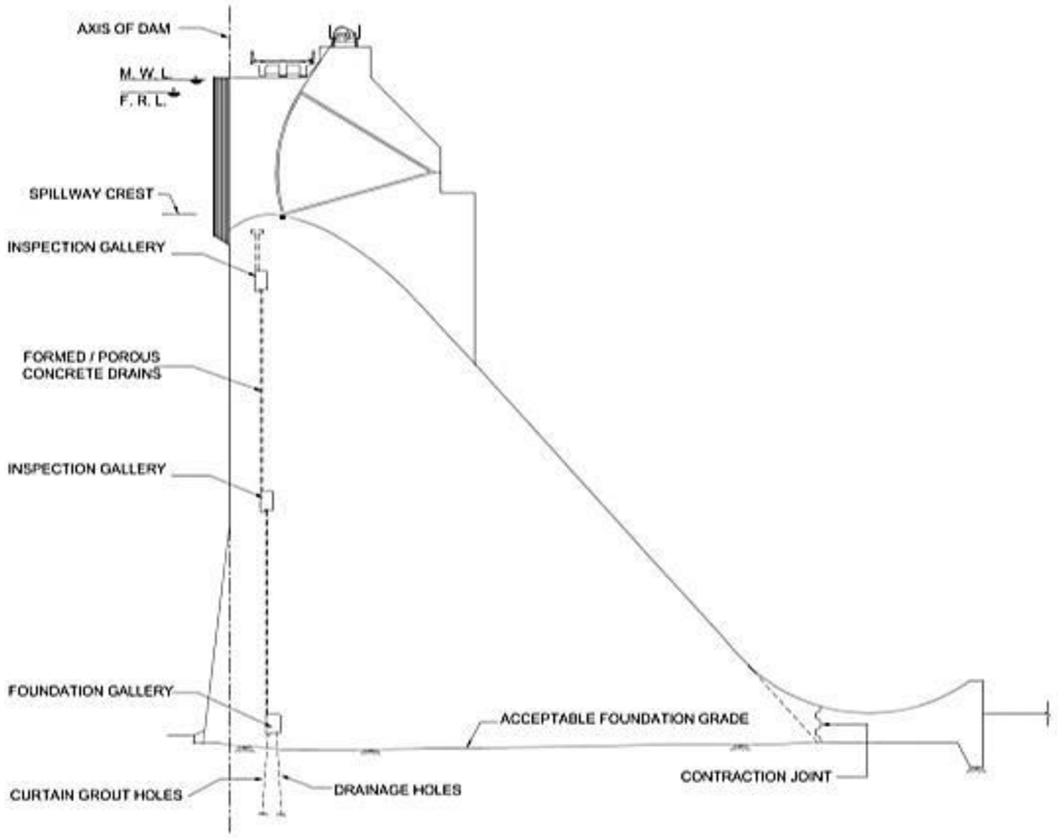

Figure 6-2: Typical Overflow (Spillway) section of a Gravity dam

The spillway (overflow) section is generally provided with a curved crest shape approximating the lower nappe of flow over a sharp crested weir. The slope of the downstream face (i.e. spillway glacis) is made tangent to the spillway crest profile at the top and also to the curve at the junction with the stilling basin or a bucket at its bottom. Spillway piers are provided to support the bridge over the spillway and for gated spillways to support the gates also.

For overflow sections the base width is generally determined by extending the spillway downstream slope up to the foundation grade level. Also a vertical longitudinal contraction joint is normally provided close to this intersection point with the foundation rock on its downstream, to separate out the downstream energy dissipation structures from the main spillway dam structure. If a vertical longitudinal joint is not provided then the concrete mass downstream of this intersection point must also be investigated for internal stresses.

River sluices where provided to release waters for environmental purposes or for depletion of a reservoir during emergency or to release flows downstream for irrigation to be drawn from canals taking off from a pickup weir located downstream of the dam or for any other purposes are often placed in the spillway monoliths as this way a separate energy dissipation arrangement can be avoided.

6.2.3 Foundation Treatment

Many Gravity dams have been constructed on complex geological set ups containing features like shear zones, faults, shear seams, highly jointed rock etc. These features require proper foundation treatment so that issues like stress transfer, sliding stability, seepage or piping are adequately addressed. Extensive foundation treatment in such cases is not uncommon e.g. Bhakra dam, Sardar Sarovar dam, Salal dam etc.

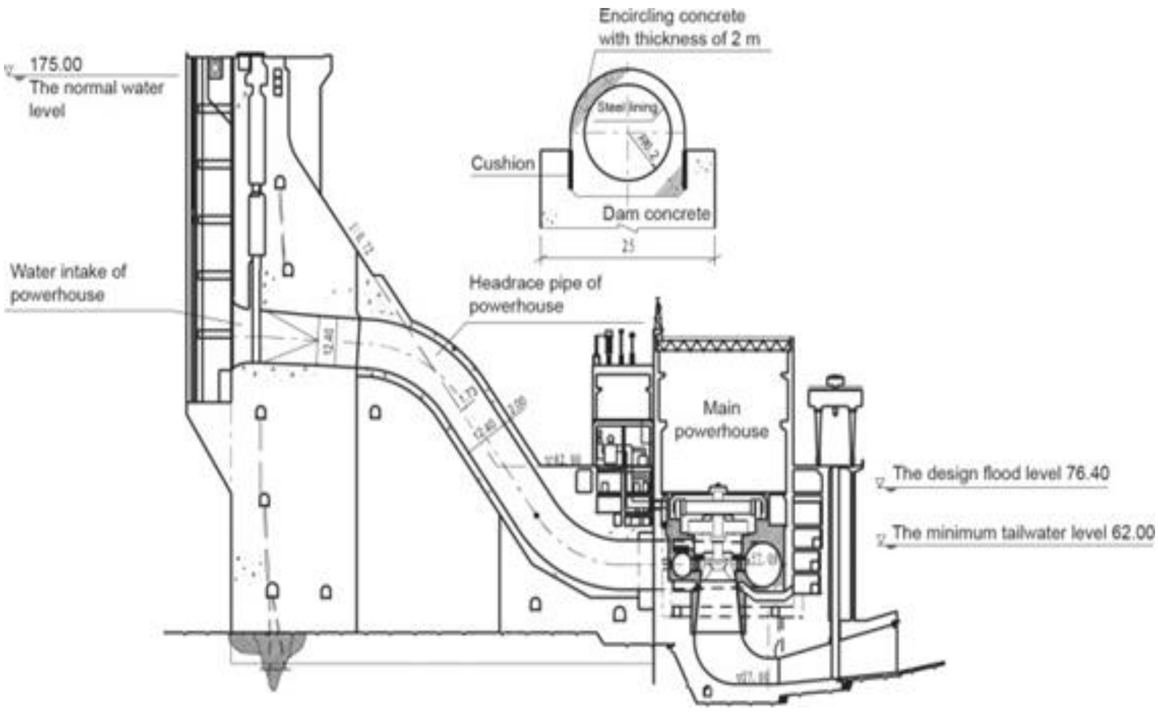

Figure 6-3: Typical Power dam section

Also some relatively low head dams have been constructed on piling in some countries. Such foundations require special provisions for seepage control and/or sliding resistance.

In general, foundation excavation profiles should be shaped so that a uniformly varying profile is obtained free of sharp offsets or breaks.

Consolidation grouting is performed to fill voids, fracture zones, and cracks in the foundation immediately below the excavated surface. In weak rock, the base of the dam usually is placed deeper into the foundation, and resistance to sliding usually can be increased by sloping the foundation surface downward in the upstream direction. When horizontal or near-horizontal stratifications exist in the foundation, the base of the structure, where feasible, should be located on the stronger strata, even if excavation in the weaker strata is necessary. On abutment slopes the steps or changes in the base elevations should be located at monolith joints. IS 11155 – Construction of Spillways and similar structures – code of practice provides guidance on shaping of the foundation grade profile for Gravity dams.

Although it is customary to curtain grout and drain the foundations, it is not uncommon to find one or both of these features absent in many existing dams. Sometimes the absence of a deep grout curtain is due to the impermeability of the foundation. However these foundations still should be provided with a drainage curtain. At some old dams, silt deposits on the reservoir floor adjacent to the dam and the natural drainage conditions in the rock have combined to preclude the existence of high uplift pressures.

At the same time there are many examples of dams in which the foundation drainage holes are choked which are needed to be reopened/re-drilled. In most cases, the stability of these old dams can be substantially improved by drilling fresh foundation drains/making functional the existing foundation drains.

Other possible causes of concern in existing dams may be daylighting of certain shear seams/weak features on account of scouring below the spillway which will require examination of sliding stability of the dam and looking into possible remedial measures, if required.

6.2.4 Galleries and Adits

A system of galleries, adits, chambers, lift and stair wells are normally provided in all Gravity dams (there are some old dams without a foundation gallery also). They provide means of access and space for drilling and grouting, for provision of grout curtain and drainage holes for draining the seepage through both the foundations and the dam body, collection of this seepage water in sump wells and its disposal by pumping/gravity; and the installation, operation, and maintenance of the accessories and utilities of the dam. These openings also provide access for inspecting the interior. Their extent depends upon the size of the dam and project functions. For general requirements of galleries and other openings in dams, IS 12966 (Part 1) may be referred to.

A grouting and drainage gallery normally should extend the full length of the dam. As it is located near the foundations it is called foundation gallery. As per IS 12966 (Part1) foundation gallery should be provided in a dam in which the dam height measured from foundation level is more than 10 m (measured up to the crest level in case of overflow section). For dam height less than 10 m its necessity has been left to the discretion of the designer. It should be located near the upstream face and as close to the foundation surface as feasible.

As per USBR a clear distance equal to at least 5% of the reservoir head subject to a minimum of 5 ft. (1.5 meter) is usually kept between the upstream face of the dam and the gallery to allow room for concrete placement and compaction, and to minimize the possibility of cracking which can be associated with serious leakage from the reservoir. Further the USBR specifies a minimum of 5 ft. (1.5 meter) of concrete between the floor of the gallery and the foundation. The provisions contained in IS 12966 (Part1) in this regard are generally quite similar except that it stipulates that the minimum distance of the upstream face of gallery from the upstream face of the dam should not be less than 3 m instead of 1.5 m as prescribed by USBR.

The minimum size of a gallery as per the above Indian Standard is 1.50 m x 2.25 m. However it is also recommended that a larger size of 2.0 m x 2.5 m can also be provided to accommodate the drilling equipment. The floor of the gallery should slope towards the gutter along the upstream side, into which all the seepage water is discharged. The gallery is arranged as a series of horizontal runs and stair flights to follow the longitudinal foundation line.

In high gravity dams, a second drainage gallery is some- times provided about one-half to two-thirds of the base width downstream of the grouting and drainage gallery.

Also intermediate inspection galleries should be provided at about every 30 m intervals of size 1.50m x 2.25 m. in case of high dams. This is to facilitate cleaning/reaming of the formed/porous concrete drains in the body of the dam.

The sizes of gate chambers, located directly over service and emergency sluice gates, are determined by the sizes of gates and hoists. Access galleries should be of sufficient size to permit passage of the largest component of the gates and hoists.

Other types of galleries include visitors' galleries to allow the public into points of interest, cable galleries to carry control or power cables, and galleries solely for inspection or to accommodate instrumentation.

For other details IS 12966 (Part1) can be referred to.

The health of an existing dam may or may not be ascertained from the condition of its galleries. The condition of the gallery might be very deceiving. A dry gallery might indicate that there is no seepage and thus no uplift pressures; however, the drains could be plugged and uplift pressures have increased. A wet gallery might indicate the drains are working properly and uplift pressures are being relieved; however, conditions may have changed and new seepage has started indicating a problem in the foundation. The best scenario is a historic record of the conditions in the gallery, measurement of the seepage flow rate, indication that there is no sediment transport in the seepage flow, and uplift pressure measurements. All these are important indicators that should be taken into account.

Seepage problems, choking of drains etc. are quite common and require considerable planning and rehabilitation efforts. This has been discussed in details in the Manual for Rehabilitation of large dams (Doc. No. CDSO_MAN_DS_02_v 1.0).

Further IS 11216 – 1985 – Code of practice for permeability test for Masonry during and after construction prescribes a water loss of not more than 2.5 and 5 lugeon in the u/s and d/s portions of the Masonry dam respectively.

6.2.5 Contraction Joints

There are basically two types of contraction joints in a dam viz. Longitudinal (parallel to the dam axis) and Transverse (upstream to downstream) contraction joints.

There are only a very few cases of Gravity dams with longitudinal contraction joints in India e.g. Bhakra dam.

Further it is recognized that the practice of dividing a monolith into two or more blocks by introducing joints parallel to the dam axis is basically unsound unless high degree of perfection is accomplished in ensuring monolithicity by provision of suitable shear keys and successfully grouting at the appropriate time.

As such by and large only transverse contraction joints are being provided in new Gravity dams. However, if an existing dam has longitudinal joints, they must be considered in the stability analyses. Transverse vertical contraction joints can be: filled with cement grout, left open with a gap between monoliths, be smooth, or contain shear keys. The characteristics of these joints should be determined as this will affect the type of stability analyses to be performed and the assessment of the stability of the dam.

As per IS 6512 transverse contraction joints can be provided at a spacing of 15-25 m in concrete dams and a larger spacing for masonry dams.

Water stops are provided at the contraction joints as per IS 12200 and IS 15058. The typical arrangement followed will vary from dam to dam. The water stops normally used include Copper or Monel water stop, Asphalt water stop, PVC and Rubber water stops. Over the years it has been seen that the performance of asphalt water stop has not been very satisfactory. These days PVC water stops are preferred.

Failure of water stops across transverse contraction joint in any gravity dam can lead to large seepage entering the foundation gallery from the transverse pipe close to the joint location from the vertical trap drain which is usually provided at transverse joints along with the water stops. This can be observed in dam inspections as part of normal O&M of the dam.

Remedial measures for such cases are discussed in the Manual for Rehabilitation of large dams.
(Doc.No.CDSO_MAN_DS_02_v 1.0).

6.2.6 Sluices in Gravity dams

Sluices are provided for a variety of purposes such as River diversion as construction sluices which are plugged later as per project construction schedule, to supply water for irrigation purposes, municipal/industrial use, generation of hydro-power, for satisfying prior rights of downstream areas, ecological requirements, to pass flood discharge in conjunction with spillway, depletion of the reservoir in order to facilitate inspection of the reservoir rim and upstream face of the dam etc. Sluices in concrete dams are sometimes open box channels from upstream to downstream that divert river water during construction. Sluices are sometimes gated to provide controlled releases in the future or they can be plugged fully or partially with concrete and abandoned. Sometimes partial plugging is only with a few feet of concrete along the upstream and downstream openings. This can reduce the available horizontal area along a potential slide plane (i.e. along a lift joint) in the dam and should be investigated.

It is, therefore essential to periodically inspect them for any damages besides ensuring that the hydro-mechanical equipment's are working satisfactorily.

6.2.7 Lift Joints

The construction of the gravity dam is carried out in lifts to divide the structure into convenient building units and to obtain temperature regulation. These joints in conventional concrete gravity dams are usually provided at every 1.5 to 2 metres intervals. Where necessary for temperature control the lift thickness is sometimes limited to 0.75 metres in certain parts of the dam. Treatment of lift surfaces to receive the new concrete includes green cutting with an air/water jet or sand blasting. Cement slurry/mortar may be spread over the existing lift before laying of next lift for better bond and to avoid seepage through lift joints. Where the concrete has been allowed to dry out for a prolonged period, the surface is dampened with water before placing the mortar.

Roller Compacted Concrete is usually placed in lifts of 30 cm thickness. Special attention is given to mix design and/or surface condition required to assure adequate sliding resistance or water-tightness along the lift face. Special lift treatments are being adopted to control seepage through lift joints. Use of grout enriched vibratable mortar over the lift surfaces near the upstream face is one such practice which was also adopted in RCC dams constructed in India. Thicker lifts of 750-1,000 mm have been used in Japan for RCD (Roller Compacted dams).

Masonry dams are constructed in lifts not more than 60 cm thickness in one or more layers.

Any poor construction can lead to large seepage problems.

Further poor construction of these lift surfaces can also lead to lower shear resistance across such surfaces. This fact is extremely important because the strength of the lift joints is a function of concrete mix and most importantly construction techniques. To achieve bond along the lift joints, the lift joints need to be cleaned with high pressure water and cut down to aggregate during construction before subsequent concrete placements. The only true method to determine the lift joint strength is to extract concrete core through the lift joints and test in the laboratory. Acoustic or televiewer geophysical testing cannot determine the level of lift joint bond. Also, visual inspection of seepage (or no seepage) on the downstream face is not an indication of poor (or good) lift joint bond. A lift joint might be very tight and seepage free yet be unbonded.

Where found leaking (wet spots/seepage on d/s face of dam) these surfaces may require pointing on upstream face with special mortars and/or grouting of the dam body near the u/s face. However, it can be extremely difficult to stop seepage inside a gravity dam with drilling and grouting and the seepage may not be a stability issue. It is suggested stability analyses first determine if there is an issue with stability along planes in the dam due to increased uplift pressures. Then decide if additional drainage or an upstream liner or barrier is appropriate.

6.3 Requirements of Stability

As per IS 6512 the Gravity dams are required to satisfy the following requirements of stability:

i) The dam shall be safe against sliding on any plane or combination of planes within the dam, at the foundation or within the foundation.

ii) The dam shall be safe against overturning at any plane within the dam, at the base or at any plane below the base.

iii) The safe unit stresses in the concrete or masonry of the dam or in the foundation material shall not be exceeded.

6.4 Assumptions made

For consideration of stability the following assumptions are generally made as per IS 6512:

i) A gravity dam is composed of individual transverse vertical blocks (or monoliths) separated by vertical transverse contraction joints. As previously stated, if the contraction joints are smooth and open, each block carries its load to the foundation without transfer of load from or to adjacent elements. However, if the contraction joints are grouted or have shear keys, there is load transfer between adjacent blocks and this 3-dimensional effect can greatly improve the stability of gravity dams, especially in narrow canyons.

ii) The vertical stress varies linearly from upstream face to downstream face on any horizontal section. Finite element studies have shown that vertical stress in concrete dams from upstream to downstream face on a horizontal section can be higher at the faces and definitely higher at changes of geometry, but stresses can be assumed to vary linearly when using limit equilibrium methods for stability analyses. Finite element studies should be used to investigate internal stresses in the dam

Two dimensional stability analysis is normally carried out considering a vertical upstream to downstream unit width of the dam for sections that have few voids/openings. An entire monolith may have to be modeled if there are significant voids/openings in the dam, as stated below.

However in some cases it may be necessary to carry out the stability analysis for the whole block especially in case of kink blocks (non-overflow) which are tapering towards downstream (in cases when the dam axis is along two or more straight lines which meet at an angle in plan) or when there are any special features in dam or if large size openings so require.

6.5 Forces for consideration

The following forces may be considered while reviewing the stability of an existing dam:

i) Dead loads

ii) Reservoir and Tail water loads

iii) Uplift pressure

iv) Earthquake forces

v) Earth and Silt pressure

vi) Ice pressure ,if applicable

vii) Wind pressure

viii) Wave pressure

ix) Thermal loads, if applicable.

Forces listed at (i) to (v) above are invariably taken in to account while checking the dam stability.

The force at (vi) above is site-specific and depends upon the climatic conditions at dam site.

Forces at (vii) to (ix) are not so significant for stability purposes and are normally neglected.

However the forces at (vii) and (viii) above will need to be considered for designs such as parapet wall designs.

Thermal loads may need to be considered for special studies like thermal analysis.

6.5.1 Dead Loads

The dead loads comprise of:

i) Self-Weight of the dam
ii) Weights of appurtenances such as spillway pier, gates, hoists, bridge etc.

The unit weights of the materials should be adopted based on actual test results for existing dams.

However for preliminary study the unit weights of concrete and masonry can be assumed based on similar case studies/available literature taking in to account the condition of the dam.

6.5.2 Reservoir and Tail water loads

These loads are to be calculated as per IS 6512. The unit weight of water is assumed as 1000 kg/m^3. Variation in unit weight of water with temperature is usually ignored. A linear distribution of static water pressure acting normal to the face of the dam is assumed. If gates or other control features are used on the crest they are treated as part of the dam as far as application of upstream water pressure is concerned.

The weight of water flowing over the spillway crest and glacis is generally neglected as the water usually approaches sprouting velocity and exerts little pressures on the spillway crest.

As regards tail water pressure, full value of tail water depth shall be considered in respect of Non Overflow sections. However for Overflow sections, a reduced tail water depth determined at the toe of spillway section based on hydraulic design/hydraulic model studies of the spillway and energy dissipation arrangements can be considered. Full value of tail water depth should however be used while considering uplift in the stability calculations.

6.5.3 Uplift pressure

Uplift pressures due to head water and tail water loads occur as internal pressures in the body of the dam, at the dam-foundation interface and in the foundation rock.

The following assumptions are made:

i) No reduction in uplift at the downstream toe of spillway on account of the reduced water surface elevation (relative to tail water elevation) that may be expected downstream of the structure/energy dissipation arrangements.

ii) Current state of knowledge does not know how uplift pressures change un-der or within the dam during an earth-quake. As such, uplift pressures are assumed to remain the same throughout the earthquake and it was as at the start of the earthquake. However, uplift pressures can change after an earthquake depending on the damage caused by the earthquake. For instance, if a crack forms during an earthquake along the upstream face under the water surface, full reservoir head is assumed to develop in the crack after the earthquake. Also, the shear strength is assumed to reduce in an earthquake-induced crack. A post-seismic stability analyses of the dam is then suggested with increased uplift and reduced shear strength.

6.5.3.1 Un-cracked sections

Uplift pressures are assumed as acting over the entire 100% of the area of the plane considered. As per IS 6512 they are to be calculated for the following two cases, namely:

i) Normal Uplift

ii) Extreme Uplift

The normal uplift shall correspond to the condition when the foundation/body drains are functional/operative whereas extreme uplift to the condition when they are inoperative/choked.

At the line of drains, IS 6512 recommends an uplift intensity equal to the tail water head plus one-third the difference between the upstream reservoir level and the corresponding tail water level for new dams, in respect of normal uplift.

Practices of organizations like the United States Bureau of Reclamation, US Army Corps of Engineers etc. regarding uplift can be seen from their publications for reference. The 2/3 pressure reduction at the line of drains is an average pressure measured at many Bureau of Reclamation concrete dams. At Reclamation, the drains are typically at

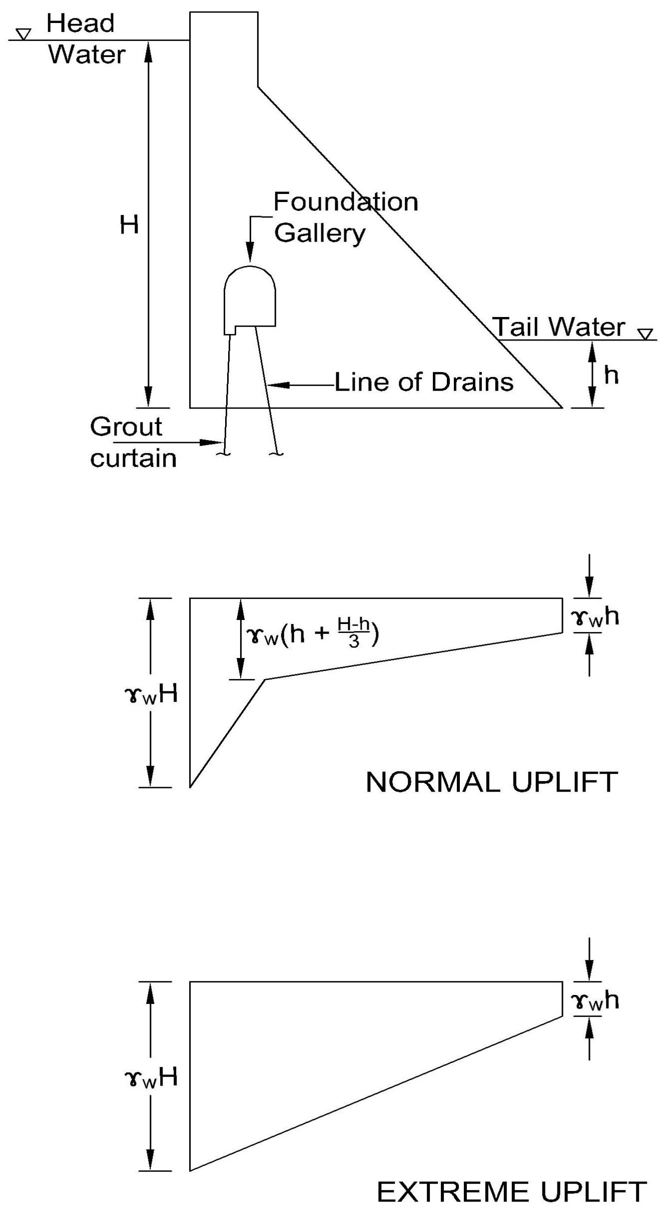

Figure 6-4: Typical normal and extreme uplift pressure distribution

least 2-inch diameter, spaced at 10-feet apart, and are positioned at 5% of the dam height away from the upstream face. The Corps of Engineers uplift equations allow more flexibility in the drain location and gallery elevation. Standard practice is to not allow for any reduction of uplift pressures at the line of drains if there are no uplift measurements.

At the upstream heel the uplift pressure is assumed to be equal to the head corresponding to the upstream reservoir level and at downstream toe it is assumed to be equal to the head corresponding to the tail water level.

The uplift pressure distribution in case of normal uplift shall vary linearly from the uplift intensity assumed at the line of drains to the reservoir level on the upstream heel and to the tail water level on the downstream toe.

In case of extreme uplift, the uplift pressure distribution shall be assumed to vary linearly from the reservoir level at the upstream heel to the tail water level on the downstream toe.

The normal and extreme uplift pressure distribution normally adopted in stability analysis is shown at Figure 6-4.

In case of existing dams the actual uplift pressure distribution can be ascertained from uplift pressure cells where they are installed.

Where the dam body/foundation drains are seen to be choked with calcinations it is necessary to ream/re-drill them to make them functional.

6.5.3.2 Cracked Sections

IS 6512 does not allow for any cracking in Gravity dams. However, eliminating all cracking in existing dams during a large earthquake may not be possible, especially at locations of changes in geometry and especially since many were not designed considering current earthquake levels. Fortunately, concrete gravity dams have performed historically very well during large earthquakes with minimal damage. As such, it is suggested post-seismic analysis be performed to make sure the concrete dam is stable after the earthquake.

Organizations like USBR or the US Army Corps of Engineers do not allow cracking (vertical tension) along the upstream face of a concrete dam during Usual load conditions (normal operating conditions), but do allow cracking during Unusual (flood) or Extreme (seismic) loading conditions if the dam remains stable.

As such for existing concrete gravity dams (not in masonry dams) it may be considered to allow cracking on a case to case basis as long as the compressive stresses after cracking are within permissible limits and the sliding stability is satisfied considering uncracked width of the dam in respect of certain conditions like Load Combination F (Reservoir at MWL with drains chocked) and Load Combination G (Reservoir at FRL with earthquake and drains choked).

It may also be desirable to check that the stresses obtained in such cases by static/dynamic finite element analysis (as applicable) remain within permissible limits.

However in case cracks are observed in a dam after high flood or an earthquake event or if the seepage quantity is found to increase, it will be necessary to seal them by grouting.

Details are covered in para 6.8.2.

During an earthquake, assume the uplift pressure remains the same throughout the earthquake as it was at the start of the earthquake.

6.5.4 Earthquake forces

The seismic zone for an existing project is to be determined as per the Seismic map of

India given in IS 1893 – 2002. The seismic co-efficient to be used in pseudo-static analysis can be calculated as prescribed in IS 1893 – 1984. Where recommendations of National Committee on Seismic Design Parameters (NCSDP) are available, they may be adopted. The vertical seismic co-efficient is normally taken as 2/3 rd of the horizontal seismic co-efficient. For pseudo-static analysis the hydro-dynamic pressures and inertia forces can be calculated in accordance with IS 1893 – 1984. Some of the codal provisions in this regard are given below for ready reference.

6.5.4.1 Hydrodynamic Effects due to reservoir

Hydrodynamic interaction is the seismic effect of water pressures acting on the concrete dam. Historically methods developed by Westergaard in 1933 (vertical face) and Zangar in 1952 (sloping faces) are used. The methods involve computing a mass of water affected by the movement of the dam times the acceleration of the dam to account for the inertia force of the water. These methods, as presented below, are widely used but should only be considered in preliminary stability of the concrete dam. More advanced methods to compute hydrodynamic interaction should be considered if hydrodynamic forces are an issue.

Earthquake excitation causes hydrodynamic pressure (or suction) exerted against the dam in addition to hydro- static pressures. Based on the assumption that water is incompressible, the hydrodynamic pressure at depth y below the reservoir surface shall be determined as follows:

$$p = C_s \alpha_h w h$$

where,

p = Hydrodynamic pressure in kg/m^2 at depth y,

C_s = Coefficient which varies with shape and depth.

α_h= Design horizontal seismic coefficient

w = Unit weight of water in kg/m^3, and

h = Depth of reservoir in m.

The variation of the coefficient Cs, with shapes and depths, is illustrated in Appendix G of IS: 1893-1984. For accurate determination, these values may be made use of. However, approximate values of Cs for dams with vertical or constant upstream slopes may be obtained as follows:

$$\boldsymbol{C_s = \frac{C_m}{2}\left\{\frac{y}{h}\left(2-\frac{y}{h}\right)+\sqrt{\frac{y}{h}\left(2-\frac{y}{h}\right)}\right\}}$$

where,

C_m = Maximum value of C_s obtained from Fig. 6-5,

y = Depth below reservoir surface, and

h = Depth of reservoir

For dams with combination of vertical and sloping faces, an equivalent slope may be used for obtaining the approximate value of C_s. The equivalent slope may be obtained as explained below.

If the height of the vertical portion of the upstream face of the dam is equal to or greater than one-half the total height of the dam, it can be taken as if it is vertical throughout. If the height of the vertical portion of the upstream face of the dam is less than one-half the total height of the dam, use the pressure on the sloping line connecting the point of intersection of the upstream face of the dam and the reservoir surface with the point of intersection of the upstream face of the dam with the foundation.

The approximate values of total horizontal shear and moment about the center of gravity of a section due to hydrodynamic pressure are given by the following relations:

V_h = 0.726 py

M_h = 0.299 py2

where,

V_h = Hydrodynamic shear in kg/ m at any depth, and

M_h = Moment in kg-m/ m due to hydrodynamic force at any depth y.

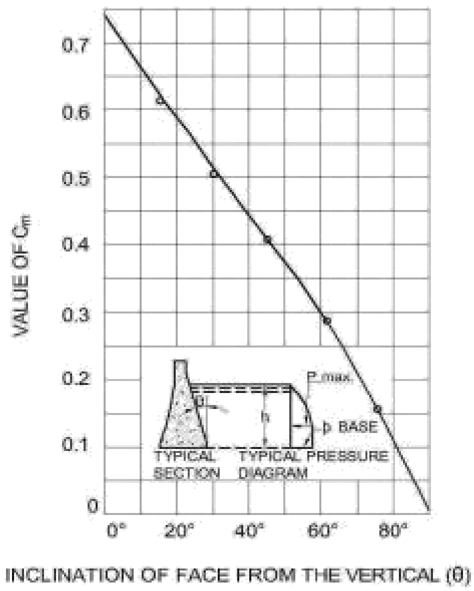

Figure 6-5: Maximum Value of Pressure Coefficients (C_m) for Constant Sloping Faces.

6.5.4.2 Effect of Horizontal Earthquake Acceleration on the Vertical Component of Reservoir and Tail Water Load

Since the hydrodynamic pressure acts normal to the face of the dam, there shall, therefore, be a vertical component of this force if the face of the dam against which it is acting is sloping, the magnitude at any horizontal section being:

$$\boldsymbol{W_h = (V_1 - V_2)\tan\theta}$$

where,

W_h = Increase (or decrease) in vertical component of load in kg due to hydrodynamic force,

V_2 = Total shear in kg due to horizontal component of hydrodynamic force at the elevation of the section being considered,

V_1 = Total shear in kg due to horizontal component of hydro- dynamic force at the elevation at which the slope of the dam face commences, and

θ = Angle between the face of the dam and the vertical.

The moment due to the vertical component of reservoir and tail water load may be obtained by determining the lever arm from the centroid of the pressure diagram.

6.5.4.3 Inertia Forces

For concrete and masonry gravity dams, the inertia forces shall be considered in addition to the hydrodynamic pressures.

For dams up to 15 m height the horizontal seismic coefficient shall be taken as 1.5 time's seismic coefficient αh at the top of the dam reducing linearly to zero at the base. Vertical seismic coefficient shall be taken as 1.5 times the value of αv at the top of the dam reducing linearly to zero at the base.

Charts for calculating horizontal & vertical inertia forces on various geometrical areas/shapes for use in Seismic Coefficient Method are given in Appendix-B for ready reference.

Alternatively the dam section can be divided into a large number of slices and inertia forces calculated for each slice by considering the acceleration values at the center of gravity of each slice and subsequently summed up.

For dams over 15 m height the response spectrum analysis (RSA) procedure summarized in Section 6.12.4 and presented in detail in Appendix – C is recommended.

However both the seismic coefficient method (for dams up to 15 m height) and RSA procedure (for dams greater than 15 m in height) are meant only for preliminary review of dams. For final evaluation, dynamic response history analysis procedures are recommended. (Sections 6.12.3 and 6.12.6)

6.5.4.4 Effect of earthquake acceleration on uplift forces

Effect of earth- quake acceleration on uplift forces at any horizontal section is determined as a function of the hydrostatic pressure of reservoir and tail-water at the upstream and downstream faces of the dam. During an earthquake the water pressure is changed by the hydrodynamic effect. However, the change is not considered effective in producing a corresponding increase or reduction in the uplift force. The duration of the earthquake is too short to permit building up of pore pressure in the concrete and rock foundations.

6.5.4.5 Effect of earthquake acceleration on silt loads

Typically the dynamic forces during an earthquake from silt in the reservoir on the concrete dam can be ignored if the silt is not very deep. However, silt can be a significant contributor of hydrodynamic force on the dam if the silt is deep or if the dam is relatively thin.

6.5.4.6 Earthquake Forces for Overflow Sections

The provisions for the dam as given above will be applicable to Over-flow sections as well. In this case, the height of the dam shall be taken from the base of the dam to the top of the spillway bridge for computing the period as well as shears and moments in the body of the dam. However, for the design of the bridge and the piers, the horizontal seismic coefficients in either direction may be taken as the design seismic coefficient for the top of the dam worked out as in 6.5.4.3 and applied uniformly along the height of the pier.

6.5.5 Earth and Silt Pressures

These forces shall be calculated in accordance with IS 6512. In general the horizontal silt and water pressure is to be assumed equivalent to that of a fluid with unit weight of 1360 kg/m^3 and the vertical silt and water pressure as equivalent to that of a fluid with unit weight of 1925 kg/m^3.

As the water pressure is considered separately, the horizontal silt pressure is calculated using the balance unit weight of 360 kg/m^3 and the vertical silt pressure with the balance unit weight of 925 kg/m^3.

6.5.6 Ice pressure, Wind pressure Wave pressure and Thermal loads

These forces, where relevant, can be estimated as per IS 6512/ specialized technical literature.

6.6 Load Combinations

The Load Combinations prescribed in IS 6512 hold good for existing dams also.

Depending on the condition of the dam and reservoir operation, all the Load Combinations may not be applicable ipso-facto to an existing dam and suitable modifications may be required to be made.

The Load Combinations normally considered are listed below:

i) Load Combination A (Reservoir Empty Condition) – No reservoir and no tail water

ii) Load Combination B (Normal Operating Condition) – Reservoir at full reservoir elevation (FRL), normal dry weather tail water, normal uplift, silt, ice (if applicable).

iii) Load Combination C (Flood Discharge Condition) – Reservoir at maximum flood pool elevation (MWL), all gates open, tail water at flood elevation, normal uplift, and silt.

iv) Load Combination D – Load Combination A with earthquake.

v) Load Combination E – Load Combination B with earthquake.

vi) Load Combination F – Load Combination C but with extreme uplift (drains inoperative or if there is no uplift measurements to verify the drain effectiveness)

vii) Load Combination G – Load Combination E but with extreme uplift (drains inoperative)

viii) Load Combination H - Post-seismic with consideration of postulated damage that occurs to the dam during an earthquake from cracking causing increased uplift and reduced shear strength.

Normally load combinations A and D may not be applicable in an existing dam.

However any other Load Combination which is considered relevant for a particular existing dam may also be examined.

One of the most common deficiencies noted in Gravity dams is that of the dam body/foundation drains being choked due to leaching, poor maintenance or due to any other reason and such a situation continuing for years.

In such cases the drains are required to be taken as choked for calculating uplift and normal uplift needs to be replaced by extreme uplift in Load Combinations B, C and E as well.

In some cases, dynamic analysis for Maximum Credible Earthquake/Design Basis Earthquake may be necessary.

Where there is cracking/deterioration in dam on account of seasonal/annual temperature variations, finite element analysis of the dam with thermal loading may be required.

Deterioration from thermal effects mainly include freeze-thaw damage that can reduce the thickness of a gravity dam. Typically the stability is performed using a reduced dam thickness caused by freeze-thaw deterioration.

Typical stability analysis performed using excel programs is attached at Appendix – A for ready reference.

6.7 Stability Criterion

6.7.1 Resistance against overturning

Before a Gravity dam overturns bodily, other types of failure occur such as cracking at the upstream due to tensile stresses, increase in uplift, crushing of downstream toe material, sliding etc. A Gravity dam is therefore considered safe against overturning if the permissible tensile and compressive stresses in the dam and foundations are not exceeded and if it is safe against sliding. As such IS 6512 does not prescribe any separate check exclusively for overturning.

6.7.2 Sliding Stability

Currently two methods are used to evaluate sliding stability:

(1) Shear friction method

(2) Limit equilibrium method.

The first method i.e. the shear friction method defines the sliding factor of safety as the ratio of the total available sliding resistance (due to both friction and cohesion) along the plane under consideration to the sum of forces tending to produce sliding. Sliding can be checked by considering available sliding resistance due to friction only and also by considering available sliding resistance due to both friction and cohesion. In fact this was the practice followed in the earlier version of IS 6512 (Year 1972 edition).

The USBR and Bureau of Indian Standards have both adopted this method.

Further different partial factors of safety can also be applied separately to both friction and cohesion using this approach. This has been adopted in IS 6512 in its current edition.

The second method i.e. the limit equilibrium method defines the factor of safety against sliding as the ratio of shearing strength to the applied shear stress and has been adopted by the US Army Corps of Engineers. This method is particularly useful when wedge stability is required to be done.

Critical to the use of both methods in the investigation of sliding at or below the base, are the foundation shear strength parameters. These values must be established by appropriate laboratory or in-situ strength tests on representative foundation specimens coupled with a thorough information regarding the geology of the rock foundations, non-homogeneities/weak features in the foundations and experienced judgment. The adverse orientation of any weaknesses and its continuity must be identified and analyzed.

In India the factor of safety against sliding is determined as per IS 6512 – 1984. Partial factors of safety in respect of friction ($F_Ø$) and cohesion (F_c) are used. Their values are given in Table 6-1 below.

The factor of safety against sliding is computed from the following equation and it shall not be less than 1.0.

$$\boldsymbol{F} = \frac{\dfrac{\sum(\boldsymbol{W}-\boldsymbol{U})\boldsymbol{tan\emptyset}}{\boldsymbol{F_\emptyset}} + \dfrac{\boldsymbol{c.A}}{\boldsymbol{F_c}}}{\boldsymbol{P}}$$

where,

F = Factor of Safety against Sliding

W = Total Vertical load

U = Total Uplift force

tan Ø= Co-efficient of internal friction of the material

c = Cohesion of the material at the plane considered

A = Area of plane under consideration for cohesion. Only the area under compression is to be considered in dams in which cracking is proposed to be allowed in an existing dam.

$F_Ø$ = Partial factor of safety against friction

F_c = Partial factor of safety against cohesion

P = Total Horizontal force

For existing dams the shear parameters c and Ø may be taken from the past design records of the dam. Where they are not available, then for preliminary studies they may be assumed on the basis of available data on similar or comparable materials. Conservative values need to be assumed. Where feasible, fresh laboratory and field tests may be carried out.

Where the sliding stability is to be checked along two/three planes in the foundation envisaging wedge stability, International publications like EM-1110-2-2200 (June 1995) of the US Army Corps of Engineers on Gravity dam design, standard books like Advanced Dam Engineering by Robert Jansen and relevant technical papers in various ICOLD proceedings and other technical journals can be referred to. Such problems are often encountered in dams founded on complex geological set ups for checking against sliding along weak planes like shear seams, shear zones, faults etc. in the foundation rock mass. These features may be horizontal/near

Table 6-1: Partial Factors of Safety against Sliding

Sl. No.	Loading Condition	FØ	Fc: For dams and the contact plane with foundation	Fc: For Foundation – Thoroughly Investigated	Fc: For Foundation – Others
1	A,B,C	1.5	3.6	4.0	4.5
2	D,E	1.2	2.4	2.7	3.0
3	F,G	1.0	1.2	1.35	1.5

horizontal or dipping in the upstream or downstream directions.

In case of existing spillway dam blocks the problems can get aggravated on account of erosion/scouring due to surplus waters flowing over them and resulting in daylighting of such geological features and loss of available passive resistance on the downstream. In Non-Overflow blocks these problems may be relatively less. Foundation Treatments like pre-stressing, provision of shear keys along such features, drainage provisions etc. may sometimes need to be considered in case the factor of safety against sliding is less than permissible.

6.7.3 Compressive Strength

6.6.3.1 Concrete

The compressive strength of concrete can be determined from the cores taken out from the existing dam.

IS 6512 recommends that the compressive strength of concrete should be at least 4 times the maximum computed stress (principal stress) or 14 N/mm^2 whichever is more.

This factor is normally used for the normal loading condition. For other load conditions like spillway functioning condition at design flood or earthquake conditions a lower factor can be assumed.

6.6.3.2 Masonry

The compressive strength can be determined by compressing to failure cylinders cored out of the existing masonry dam for the purpose.

IS 6512 recommends that the compressive strength of mortar of masonry should be at least 5 times the maximum computed stress (principal stress) or 12.5 N/mm^2 whichever is more.

This factor is normally used for the normal loading condition. For other load conditions like spillway functioning condition at design flood or earthquake conditions a lower factor can be assumed.

Where required a co-relation may be established between the strength of mortar and masonry using appropriate size specimens.

6.7.4 Tensile Strength

No tensile stress shall be permitted at the upstream face of the dam for Load Combination B. Nominal tensile stresses however may be permitted in other Load Combinations as per IS 6512. Their permissible values shall not exceed the values given in Table 6-2.

However for existing concrete gravity dams (not for masonry dams), when the tensile stresses in respect of Load Combinations F and G exceed the values in the table above, it may be considered to allow cracking on a case to case basis as long as the compressive stresses after cracking are within permissible limits and the sliding stability is satisfied considering un-cracked width of the dam.

It may also be desirable to check that the stresses obtained in such cases by static/dynamic finite element analysis (as applicable) remain within permissible limits.

However in case cracks are observed in a dam after high flood or an earthquake event or if the seepage quantity is found to in-

Table 6-2: Values of permissible tensile stress in Concrete and Masonry

Load Combination	Permissible tensile Stress	
	Concrete	Masonry
C	0.01 fc	0.005 fc
E	0.02 fc	0.01 fc
F	0.02 fc	0.01 fc
G	0.04 fc	0.02 fc

crease, it will be necessary to seal them by grouting the dam body and the foundation rock.

6.8 Reaction of foundation / Base pressures

6.8.1 Uncracked sections

In the Gravity method of stability analysis, the foundation reaction is determined by the principles of statics. The resultant of all the horizontal and vertical forces should be balanced by an equal and opposite reaction at the foundation consisting of total vertical reaction and total horizontal reaction consisting of cohesive and frictional resistance at the base and resistance from passive wedge, if any. For the dam to be in equilibrium, the location of this resultant force is such that the summation of moments of all forces about any point is zero. The distribution of vertical reaction is assumed to be trapezoidal for convenience. The actual distribution of pressures however depends on the properties of dam material, foundation rock, geological features below the dam etc. A more realistic stress analysis can be performed by carrying out 2D/3D Finite Element Analysis for which specialist literature/commonly available software's could be consulted.

6.8.2 Cracked Section

6.8.2.1 Basic Considerations

In general, when allowable concrete tensile strength is exceeded, a crack is assumed to form and propagate horizontally to the point of zero stress, leaving the remaining un-cracked section entirely in compression.

Once cracking is indicated, a cracked section analysis is necessary. This involves estimation of the horizontal crack width from the upstream face and then computing the compressive stress distribution and checking for sliding considering the un-cracked width portion (See para 6.7.4).

6.8.2.2 General Iterative Method of Analysis

Once a crack forms along the upstream face of the dam, a cracked-base analysis can be performed on a horizontal plane through the dam using an iterative approach. First a crack length is assumed. The uplift profile is modified with reservoir head in the crack length and with a reduced drain efficiency using the US Army Corps of Engineers equation for uplift. Sliding is assumed to occur on the un-cracked portion of the base. Moments are computed about the center of the un-cracked base for all loads. The vertical stress in computed at the crack tip. The crack is extended longer if the stress is tensile or reduced if the stress is compressive. This process is repeated until the vertical stress at the crack tip is zero (or significantly small). This is the crack length that the dam is stable. The sliding factor of safety is then computed for the driving forces against the un-cracked portion of the base. (Reference: "Evaluation and Comparison of Stability Analysis and Uplift Criteria for Concrete Gravity Dams by Three Federal Agencies", Robert M. Ebeling et.al, Report ERDC/ITL TR-00-1, January 2000.)

6.9 Improvement in Stability of existing dams

Uplift is a significant force in the design of dams. It was not until the 1960s that the foundations under gravity dams were routinely and systematically drained to control uplift (Sims, 1994).

The stability of older dams must be judged based on in-situ investigations. Rehabilitation is expensive, and it is important to take full account of the strength of the dam and its foundations.

Stability of a gravity dam can be improved by the following methods:-

(i) By enlarging the dam section with provision of buttresses or a continuous backing of concrete/masonry on the downstream face.

(ii) By adding mass at the dam top.

(iii) By pre-stressing anchors.

(iv) By draining the dam & its foundation to reduce uplift by construction of galleries, where not provided.

Particularly in countries where earthquakes were considered as occurring with negligible frequency, it is becoming routine to review the design of older dams from the point of view of their resistance to seismic loading (BRE 1991, ICOLD 2011a).

In Germany dam safety regulations were tightened, prohibiting tensile stress at the upstream toe of gravity dams for the normal operating conditions. The response to this has been innovative work of the construction of drainage galleries in existing dams at the junction with the foundations under full reservoir (Wittke and Greb 1994) which were initially constructed without any foundation gallery. Considerable skill was exhibited in the use of a tunnel boring machine at Ennepe Dam to excavate a gallery within 3 meters of the reservoir face (Ribler and Heitefuss 1999). The work was carried out without emptying the reservoir, giving a significant benefit to the owner.

In India for strengthening of Gravity dams the following methods have been generally considered:-

i. Pre-stressing

ii. Earth backing

iii. Masonry/Concrete backing (either with continuous masonry/concrete backing or with buttresses)

Pre-stressing is considered as an emergency measure, as there is apprehension of loss of pre-stress over a period of time. Such a sit-

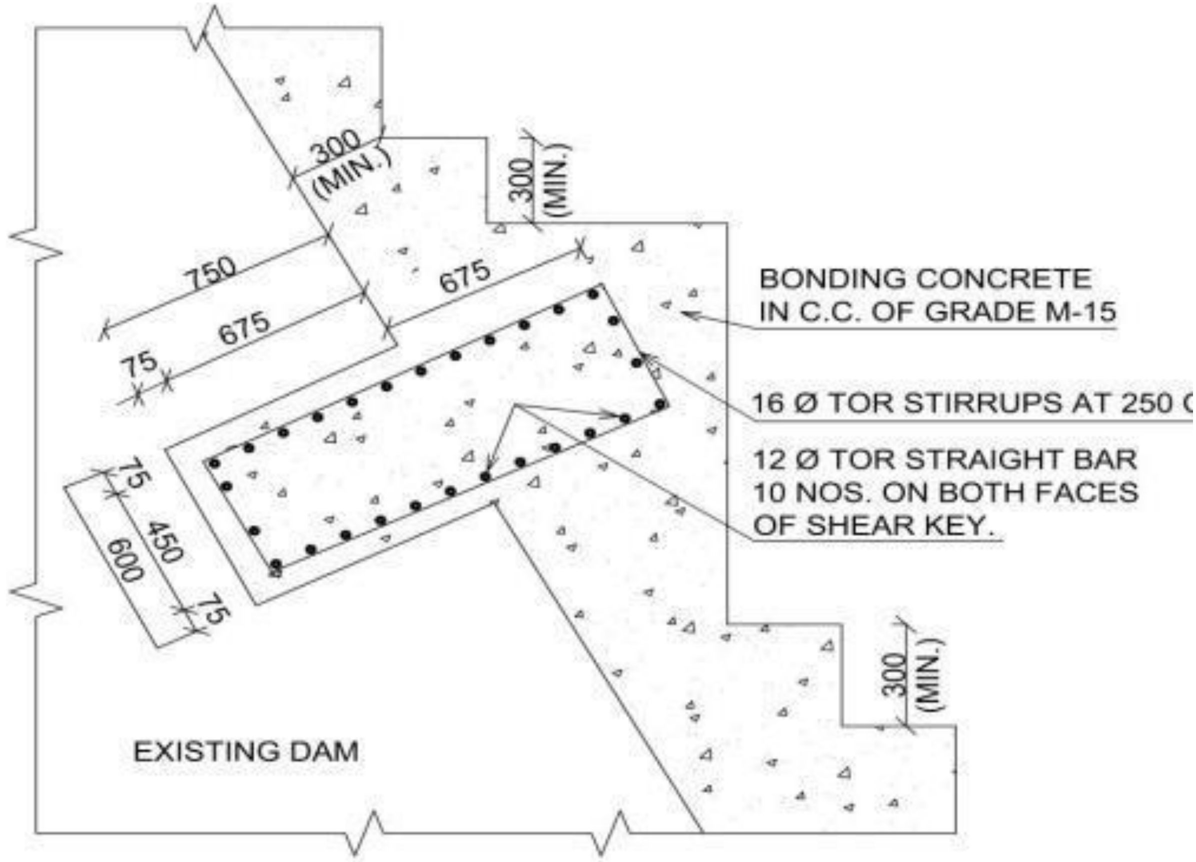

Figure 6-6: Typical details of Shear Key

uation arose in case of Bhandardara dam, wherein permanent measures such as by way of buttressing were taken up subsequently. While studying pre-stressing for strengthening of some dams, it is found that to counteract the tension developed under earthquake loading condition at the u/s heel, very close spacing of cables is required. Pre-stressing near u/s heel induces compression at u/s heel and tension at the d/s toe.

In case of downstream earth backing, there is apprehension that separation between earth and masonry can occur during earthquake, particularly near the top.

Therefore normally the feasible alternatives for strengthening have been provision of buttresses or full masonry/concrete backing. Gravity dams strengthened for earthquake loading condition are either by way of buttressing or full d/s backing. The performance of these dams so far is excellent. This measure has proved most effective and hence is generally recommended as a safe strengthening measure.

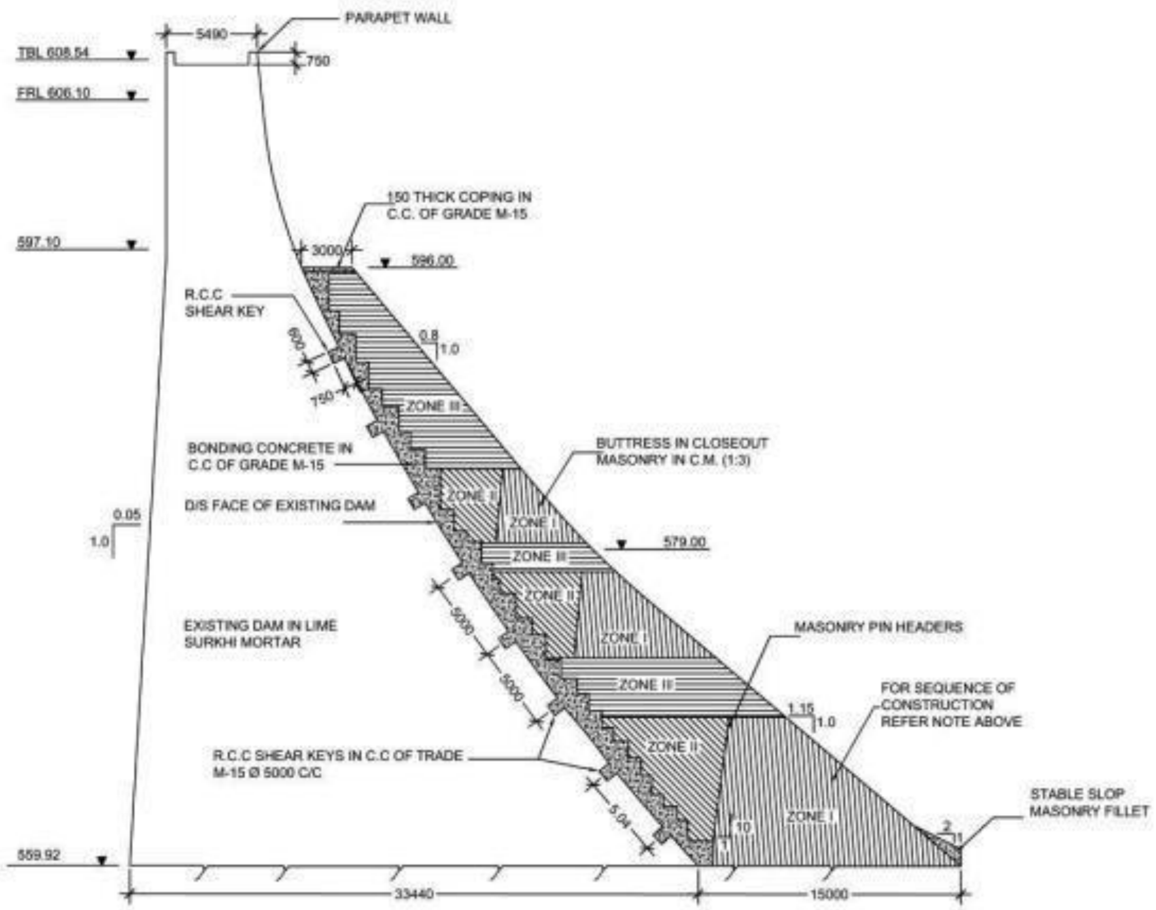

Figure 6-7: Sequence of Construction

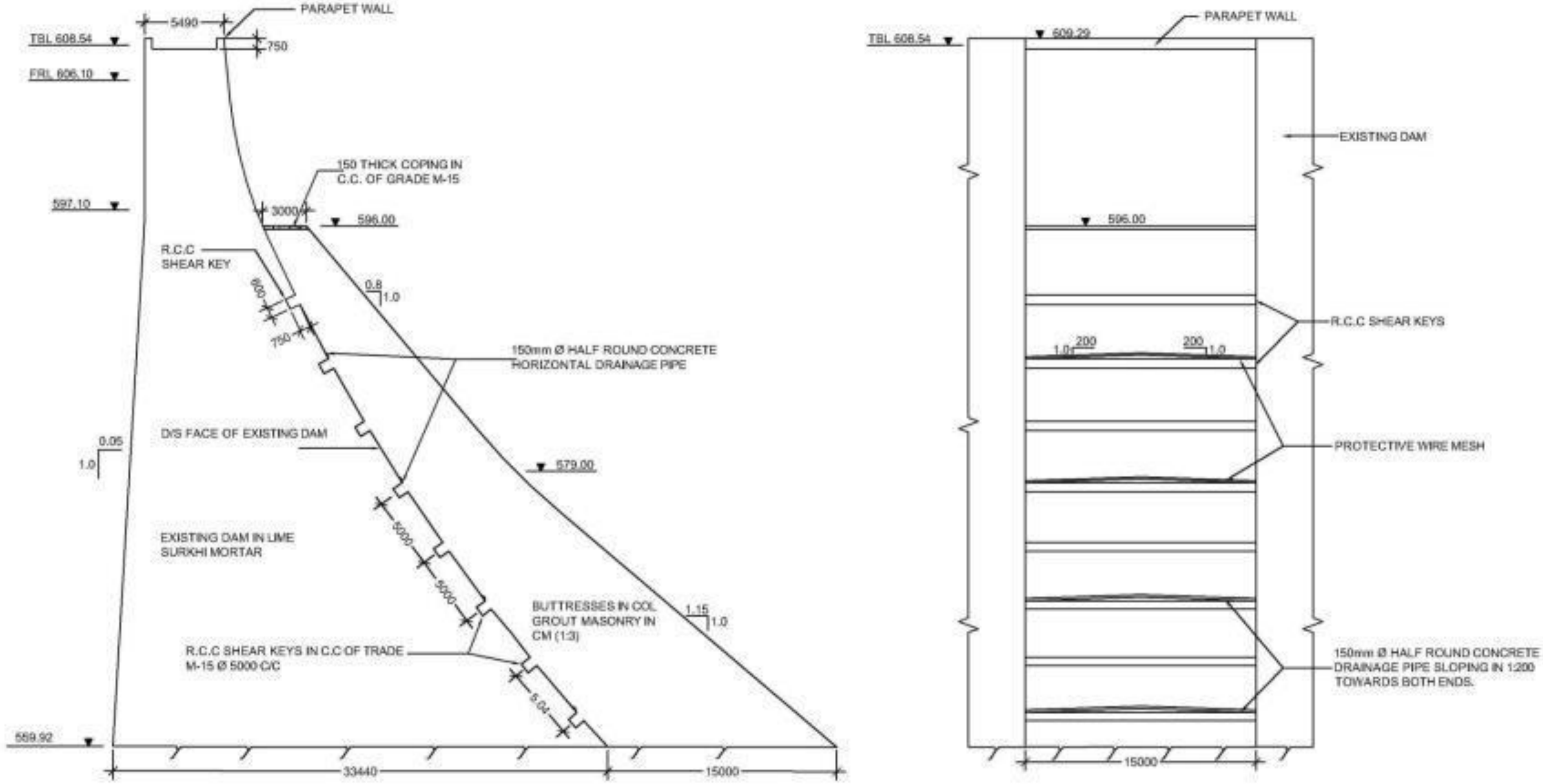

Figure 6-8: Drainage Arrangement

(Note: The change in geometry at El. 596.0 may cause stress concentrations during earthquake. The buttressing should preferably be extended to the top of the dam.)

6.9.1 Design Aspects

Water level in the reservoir at which concrete/masonry backing and the old masonry /concrete of the existing are to be bonded or joined has an important bearing on the distribution or sharing of the load by old masonry/concrete and backing masonry/concrete. The stresses corresponding to the depth of water at which bonding is done are assumed to be taken solely by the old dam. Subsequent increase or decrease in the stresses is shared jointly by the composite section. The stresses can change due to variation in reservoir level. A high bonding level allows greater flexibility in construction program but increases the backing required resulting in higher cost. Lower bonding levels place severe constraints on the time available for construction but lead to greater sharing of the load by the backing section enabling adoption of reduced sections and consequent economy.

Reservoir level at the time of bonding an existing dam with backing masonry has an appreciable effect on the locked up stresses. This has an effect on the downstream batter of the add-on masonry. Higher the reservoir level at which bonding is done, flatter will be the batter. Thus to achieve economy it is desirable to have reservoir level as low as possible at the time of bonding. The bond level also depends on the availability of time for strengthening works.

Usually the total length of buttresses is not less than half the length of dam for which strengthening is required.

While joining/bonding the old dam with the downstream buttress/continuous concrete or masonry backing, all needful precaution's like roughening the old dam surface , surface preparation/cleaning, provision of shear keys, drainage etc. are required to be taken (Figure 6-6, 6-7 & 6-8).

For more details technical paper on "Strengthening of Structures" by S.Y. Shukla and V.M.Deshpande presented in the 2nd International Conference of Dam Safety Evaluation held at Trivandrum, Kerala in November 1990 can be referred to.

Under the DRIP two Masonry dams viz. Pechiparai Dam in Tamil Nadu & Kuttiyadi Irrigation project in Kerala are being strengthened from stability considerations, mainly for seismic condition.

6.9.2 Stress Analyses

Stress analysis of gravity dams is performed to determine the magnitude and distribution of stresses throughout the dam structure for static and dynamic load conditions and to investigate the structural adequacy of the dam and the foundation. The Load combinations are outlined in para 6.6.

Gravity dams can be analyzed by either approximate simplified methods or the finite element method depending on the refinement required for the particular level of design for the dam.

For preliminary design, simplified methods like the Gravity method of stress and stability analysis are appropriate. This method is used for gravity dams in which the transverse contraction joints are neither keyed nor grouted. Most of the Gravity dams in India come under this category.

For dams in which the transverse contraction joints are keyed and/or grouted or the gravity dam is in narrow canyon, the 3-dimensional effects of the dam can be considered.

For details of these methods of analysis the USBR publication - Design of Gravity Dams (1976) can be referred to.

The finite element method is used if a more accurate stress investigation is required.

a. Finite element analysis.

Finite element models can be used for linear elastic static and dynamic analyses and for nonlinear analyses that account for interaction of the dam and foundation. The finite element method provides the capability of modelling complex geometries and wide variations in material properties. The stresses at corners, around openings, and in tension zones can be approximated with a finite element model. It can model concrete thermal behavior and can couple thermal stresses with other loads. An important advantage of this method is that complicated foundations involving various materials, weak joints, shear zones/seams, and fracturing can be readily modelled.

Two-dimensional finite element analysis is generally appropriate for concrete gravity dams. Where necessary three dimensional finite element analysis can be carried out. For long conventional concrete dams with transverse contraction joints and without keyed joints, a two-dimensional analysis should be reasonably correct. Structures located in narrow valleys between steep abutments and dams with varying rock moduli which vary across the valley are conditions that may necessitate three-dimensional modelling.

Some general purpose finite element programs are SAP, ANSYS, ABAQUS and LSDYNA.

6.10 Design Earthquakes, Target Spectra, and Selection of Ground Motions

6.10.1 Design Earthquakes and Ground Motions

Two levels of ground motion (GM) with corresponding performance requirements shall be considered in the seismic design of new dams and seismic safety evaluation of existing dams:

The Operating Basis Earthquake (OBE) is the earthquake event that produces GM at the site that can reasonably be expected during the service life of the project. This statement has usually been interpreted as GM that has a 50% probability of exceedance (PE) in 100 years, the commonly assumed life of concrete dams. The corresponding mean return period is 144 years (calculated assuming a Poisson model for occurrence of events). At this level of ground shaking, the facility-dam, appurtenant structures, equipment, power house, etc.-should experience little or no damage and continue to function without interruption; this performance requirement implies that the dam remains essentially within the linear range of behavior. The OBE should be determined by Probabilistic Seismic Hazard Analysis (PSHA).

The Safety Evaluation Earthquake (SEE) or Maximum Design Earthquake (MDE) is the earthquake event that produces GM at the site that is rare. Factors to consider in selecting the intensity of this GM are the consequences of failure of the dam, criticality of project function (power generation, water supply, flood control, etc.), and turnaround time to restore the facility to be operational after the earthquake event. The MDE represents ground shaking at the site associated with a long mean return period: 10,000, 3000, or 1000 years for dams where the consequences of dam failure are high, moderate, or low, respectively. Mean return periods of 10,000 (precisely 9950) years and 1000 (precisely 949 years) represent ground shaking associated with a 1% and 10% PE in 100 years, respectively. The MDE should also be determined by PSHA. At this level of ground shaking, there should be no catastrophic failure, such as uncontrolled release of the impounded water, although significant damage or economic loss may be tolerated. This performance requirement implies that the dam is allowed to deform significantly into the nonlinear range.

Specification of the design earthquakes by various regulatory agencies and organizations are summarized in Appendix C for convenience reference.

6.10.2 Design Spectra: Horizontal Components of Ground Motion

Uniform Hazard Spectrum.

A probabilistic seismic hazard analysis (PSHA) for a specific site determines the rate (or frequency) with which the ordinate of the pseudo-acceleration response spectrum at a vibration period of interest is exceeded. Probabilistic seismic hazard analysis integrates the relative frequencies over all conceived earthquake occurrences (on all seismic sources in the region) and GMs to calculate a combined probability of the spectral acceleration.

The UHS is constructed by implementing PSHA for spectral acceleration at each vibration period (typically for 5% damping), independent of all other vibration periods. Figure 6-9 shows the UHS with a 1% probability of exceedance in 100 years for the Pine Flat Dam site in California (119.3°W and 36.8°N). This exceedance probability corresponds to a return period of 9950 years; this is the mean time between occurrences of the specified hazard, assuming that the exceedances follow a Poisson random process. A return period of 10,000 years is often selected for critical facilities such as major dams and nuclear power plants. The UHS was determined by OpenSHA, an open-source tool (http://www.opensha.org/apps).

The UHS, which by definition has the same exceedance probability at all vibration periods is over-conservative for reasons explained elsewhere [Chopra, 2019, Section 13.1.2]. Thus, the UHS is not representative of response spectra of individual GMs expected to occur at the site and, hence, is not an appropriate target for selecting GMs to be used in dynamic analysis of dams.

Conditional Mean Spectrum. This spectrum, denoted by CMS, has been developed by researchers as a target spectrum for se-

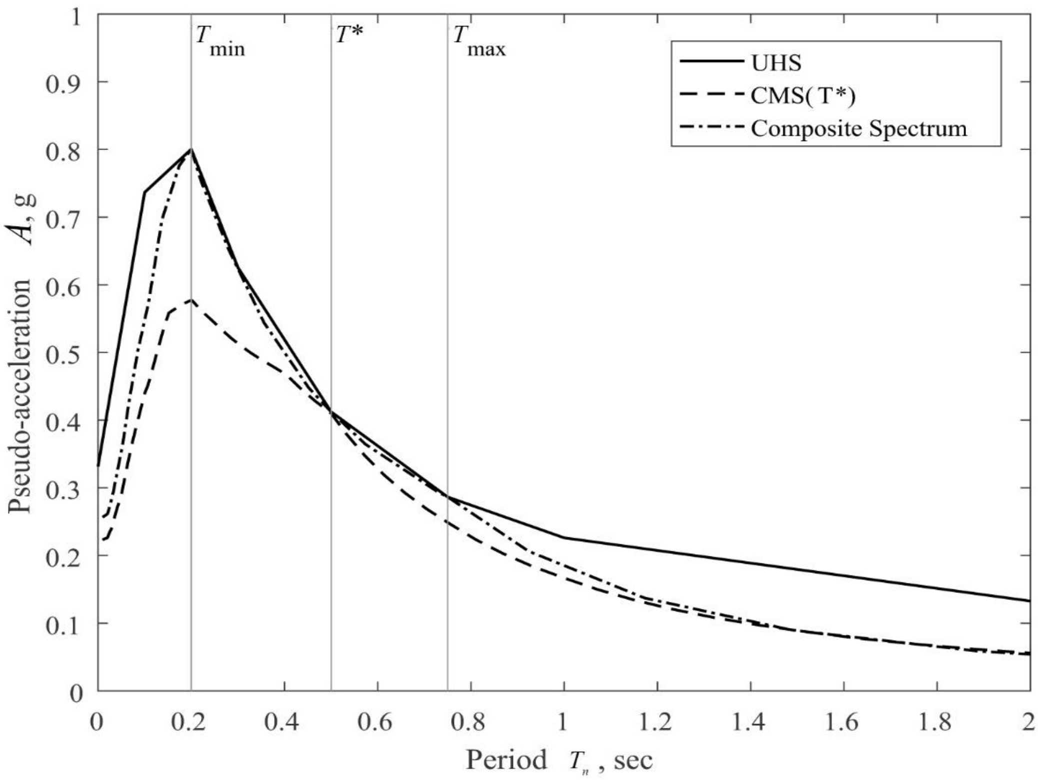

Figure 6-9: Uniform Hazard Spectrum (UHS), Conditional Mean Spectrum (CMS), with conditioning period T* = 0.5

lecting GMs that overcomes the drawbacks of the UHS. The CMS is constructed+ for a selected value of the conditioning period, denoted by T^*, where the spectral acceleration is specified. Typically, T^* is selected as the fundamental vibration period of the structure and $A(T^*)$ as the UHS value. Shown in Figure 6-9 is the CMS for the Pine Flat Dam site and T^* = 0.5 sec. It has a (slight) hump near the conditioning period of 0.5 sec where it matches the UHS and then drops off on both sides.

The CMS conditioned on a single conditioning period, T^*, say, the fundamental vibration period of the structure, is an appropriate target spectrum for dynamic analysis of structures whose response is dominated by a single mode of vibration, but not if several modes contribute significantly to the response as in the case of concrete dams [Chopra, 2019].

CMS–UHS Composite Spectrum. This spectrum has been developed for estimating seismic demands on structures where several modes of vibration contribute to the dynamic response. The CMS–UHS Composite Spectrum is defined as [Chopra, 2019, Section 13.1.4]

$$A_{\text{Composite}}\left(T_n\right) = \begin{cases} A_{\text{CMS}}\left(T^* = T_{\min}\right) & T_n \le T_{\min} \\ A_{\text{UHS}} & T_{\min} < T_n < T_{\max} \\ A_{\text{CMS}}\left(T^* = T_{\max}\right) & T_n \ge T_{\max} \end{cases}$$

where A_{CMS} is the spectral acceleration of the CMS, and A_{UHS} is the spectral acceleration of the UHS; $T_{\min}$ and $T_{\max}$ are the shortest and longest structural periods among the several vibration modes contributing to the response. As $T_{\min}$ and $T_{\max}$ approach each other, the composite spectrum reduces to a single CMS with $T^* = T_{\min} = T_{\max}$; but when the two periods are far apart, the composite spectrum is close to the UHS.

Recommended Spectra. The CMS–UHS Composite Spectrum overcomes the drawbacks of both the UHS and CMS. Therefore, this is the design spectrum recommended for selecting ground motions to be used in estimating seismic demands for concrete dams. Construction of the composite spectrum requires the UHS and two CMSs for conditioning periods $T_{\min}$ and $T_{\max}$, respectively.

The alternative is to adopt simpler approach and use the UHS as the target spectrum. This approach may seem attractive because construction of the two CMSs is avoided, but it has the disadvantage that it may lead to overly conservative estimates of seismic demands. Such over-conservatism in the analysis may lead to unnecessarily more expensive designs of new dams and to erroneous conclusions about the seismic safety of existing dams. Unfortunately, the degree of this conservatism has not been investigated.

6.10.3 Ground Motion Selection and Modification

Although the number of GMs recorded during past earthquakes is large—now reaching several thousand—this database is still not large enough to enable selection of a subset of GMs consistent with the target spectrum, especially in highly seismic regions of the world, because of the paucity of records from large-magnitude earthquakes at short distances. Thus, it becomes necessary to modify selected GM records so that their response spectra are consistent with the target spectrum. Modification of GM records usually follows one of two approaches: amplitude scaling or spectral matching. In the first approach, a GM record that is initially selected because the shape of its response spectrum is generally consistent with that of the target spectrum is scaled (usually upwards) to achieve the desired intensity; thus, the scaled record $a(t)$ = SF$a_o(t)$ where SF is

+ **MATLAB implementation of a method to compute the CMS can be downloaded from: http://www.stanford.edu/~bakerjw/gm_selection.html.**

the scale factor and $a_o(t)$ is the original record. In the second approach, a GM record is modified such that its response spectrum matches very closely the target spectrum. The amplitude scaling method is preferred because it preserves the natural variability in GM records, thus providing the mean value and an idea of the variability in seismic demands associated with an ensemble of GMs.

The goal is to select GMs whose response spectra are similar—in some sense—to the target spectrum in amplitude and in shape. The number of recorded GMs that satisfy both requirements simultaneously is often insufficient. For example, the large majority of recorded GMs are weaker than the intensity represented by the UHS and CMS in highly seismic regions. Furthermore, the response spectra for many of the records with the desired intensity $A(T^*)$ may not be similar in shape to the CMS. Given this background, selection of GMs usually proceeds in two stages. First, every record in the database is scaled to make its spectral amplitude(s) similar to the target amplitude(s). Second, the scaled records whose response spectra are similar in shape to the target spectrum over a specified period range are selected.

Ground motion selection and modification has been the subject of extensive research, originally motivated by the nonlinear response history analysis of multistory buildings [Chopra, 2017; Sections 20.8–20.14]. It is beyond the scope of this Manual to present GMSM methods in detail that is sufficient enough for users of this Manual to implement these methods. Instead, users may follow these methods, as specialized for concrete dams and extended to three components of GM, which are presented in Chopra 2019: Sections 13.2–13.5, 13.7, and 13.8.

The building engineering profession has arrived at a consensus that 11 GMs are adequate to estimate the median seismic demands to the desired accuracy. This conclusion has been based on nonlinear RHAs of a range of buildings [Chopra, 2017, Sections 20.8–20.14]. However, similar research in the context of concrete dams remains to be accomplished. In the meantime, 11 GMs are also recommended to estimate the mean (or median) seismic demands for concrete dams.

6.11 Traditional Design Procedures and Their Limitations

6.11.1 Traditional Analysis and Design

Concrete gravity dams have traditionally been designed and analyzed by very simple procedures [U.S. Army Corps of Engineers, 1958; U.S. Bureau of Reclamation 1966]. Earthquake effects were treated simply as static forces and were combined with the hydrostatic pressures and gravity loads. In representing the effects of horizontal ground motion—transverse to the axis of the dam—by static lateral forces, neither the dynamic response characteristics of the dam–water–foundation system nor the amplitude and frequency content of earthquake ground motion were recognized. Two types of static lateral forces were included. Forces associated with the weight of the dam were expressed as a product of a seismic coefficient—which was typically constant over the height, with a value between 0.05 to 0.10—and the weight of the portion of the dam being considered. Water pressures, in addition to the hydrostatic pressure, were specified as the product of the seismic coefficient and a pressure coefficient that was based on assumptions of a rigid dam and incompressible water. Finally, interaction between the dam and the foundation was not considered in computing the aforementioned earthquake forces.

The traditional design criteria required that an ample safety factor be provided against overturning, sliding, and overstressing; in particular, compressive stresses should be less than one-fourth of the compressive strength. Usually tension was not permitted, and even if it was, the allowable tension was so small that the possibility of cracking of concrete was not considered.

6.11.2 Earthquake Performance of Koyna Dam

Koyna Dam (Figure 6-10) in India was designed by the traditional static analysis procedure using a seismic coefficient of 0.05. Even though a "no-tension" criterion was satisfied in the design procedure, the earthquake of December 11, 1967 caused significant horizontal cracks in the upstream and/or downstream faces of a number of non-overflow monoliths near the elevation at which there is an abrupt change in slope of the downstream face (Figure 6-11).

The damage was repaired soon after the earthquake in two ways: first, the major cracks were repaired by injecting epoxy resin; and second, the taller non-overflow monoliths were prestressed in the vertical direction from the crest down to an elevation well below major cracks. Subsequently, it was decided to embark on a major project to strength the dam: buttresses were added on the downstream face of the non-overflow monoliths (Figure 6-12).

To understand why the damage occurred, the dynamic response of the tallest non-overflow monolith to the recorded ground motion was computed assuming linear behavior. The results indicated large tensile stresses on both faces, with the greatest values near the elevation where the slope of the downstream face changes abruptly. These computed stresses (shown in Figure 6-13), which exceeded 600 psi on the upstream face and 1000 psi on the downstream face, were about two to three times the estimated tensile strength—350 psi—of the concrete at that elevation. Hence significant cracking, consistent with what was observed after the earthquake, could have been anticipated. A similar analysis of the overflow monoliths indicated that cracking should not have occurred there, which is also consistent with the observed behavior.

Figure 6-10: Koyna Dam, India, constructed during 1954 to 1963; this dam is 103 m high and 853 m long.

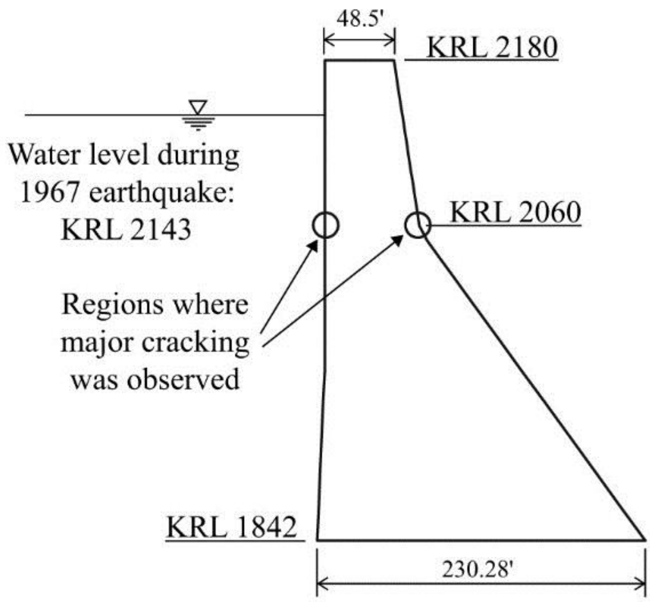

Figure 6-11: Cross section of Koyna Dam showing water level during 1967 earthquake and regions where principal cracking at the upstream and downstream faces was observed.

6.11.3 Limitations of Traditional Procedures

It is apparent from the preceding discussion that the dynamic stresses that develop in gravity dams bear little resemblance to the results obtained from traditional static design procedures. In the case of Koyna Dam, no tensile stresses were expected when designing the dam for earthquake forces based on a seismic coefficient of 0.05, uniform over the height; however, the earthquake caused significant tensile cracking in the dam. This discrepancy is the result of using too small a seismic coefficient and not recognizing the

Figure 6-12: Koyna Dam after the addition of buttresses.

amplification of acceleration over the height of the dam.

The typical design seismic coefficients, 0.05 to 0.10, as well as those recommended in Section 6.5.4, are much smaller than the ordinates of design spectra for intense earthquake motions in the range of vibration periods up to 1 sec (Figure 6-14), which exceeds the longest conceivable vibration period for a concrete gravity dam. Note that the seismic base shear coefficient values for dams are similar to those specified for multistory buildings. However, building code design provisions have been based on the premise that buildings should be able to: "(1) resist minor earthquakes without damage; (2) resist moderate earthquakes without structural damage; and (3) resist major earthquakes…without collapse but with some structural…damage." While these may be appropriate design objectives for buildings, major dams should be designed more conservatively, and this intended conservatism is reflected in the no-tension requirement imposed in traditional methods for designing dams. What the traditional methods fail to recognize, however, is that this requirement must be tied to the dynamic response of the dam that is controlled by its natural vibration periods and modes.

The effective modal earthquake forces may be expressed as the product of the weight of the dam per unit height and a seismic coefficient; its magnitude depends on the pseudo-acceleration spectral ordinate at the modal period and its height-wise distribution depends on the shape of the mode. The response of short-vibration-period structures, such as concrete gravity dams, is dominated by the fundamental mode of vibration, and the seismic coefficient varies over the dam height, as shown schematically in Figure 6-15 (b). In contrast, traditional analysis and design procedures ignore the dynamic amplification of response, as reflected in the response spectrum and the shape of the mode, and adopt a uniform distribution for the design coefficient, resulting in an erroneous distribution of lateral forces and hence of stresses in the dam. This has been demonstrated in Section 6.11.4 in the context of earthquake performance of Koyna Dam. Although triangular variation of the seismic coefficient (Section 6.5.4.3) may be an improvement over the uniform distribution, it is not consistent with vibration properties of dams (Figure 6.15 (b)).

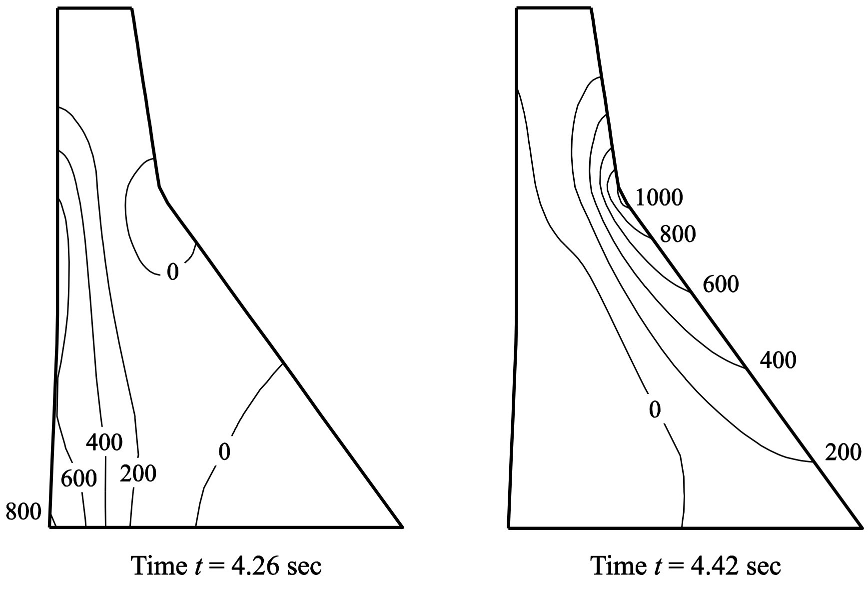

Figure 6-13: Maximum principal stresses in Koyna Dam at selected time instants due to transverse and vertical components of ground motion recorded during the December 11 1967 earthquake; initial static stresses are included.

The response results presented in Figure 6.13 also demonstrate the fallacy in the practice of decreasing the concrete strength with increase in elevation within some dams, for example, Koyna Dam [Chopra and Chakrabarti, 1973] and the 717-ft-high Dworshak Dam in the USA. This practice seems to be motivated by the observation that traditional design analyses (Section 6.11.1), which ignored the dynamics of the system, predict largest stresses near the base of the dam and decreasing stresses at higher elevations. However, as indicated by dynamic analysis (Figure 6.13) and by the location of earthquake-induced cracks in Koyna Dam, higher-strength concrete should be provided in the upper part of the dam near the upstream and downstream faces—if the designer chooses to vary the concrete strength over the dam.

The traditional design loadings for gravity dams include seismic water pressures in addition to the hydrostatic pressures, as specified by various formulas [U.S. Army Corps of Engineers, 1958; U.S. Bureau of Reclamation 1966]. These formulas differ somewhat in detail and in numerical values but not in underlying assumptions.; they are all based on the classical results [Westergaard 1933; Zangar 1952] derived from analyses that assumed the dam to be rigid and water to be incompressible. One of these formulas (Section 6.5.4.1) specifies the seismic water pressure $p_e = C_s \alpha_h w h$, where C_s is a coefficient that varies from zero at the water surface to about 0.7 at the reservoir bottom, α_h is the seismic coefficient, w is the unit weight of water, and h is the total depth of water. For a seismic coefficient of 0.1, the additional water pressure at the base of the dam is about 7% of the hydrostatic pressure; and pressure values at higher elevations are even smaller. As a result, these additional water pressures have little influence on the computed stresses and hence on the geometry of the gravity section that satisfies the traditional design criteria.

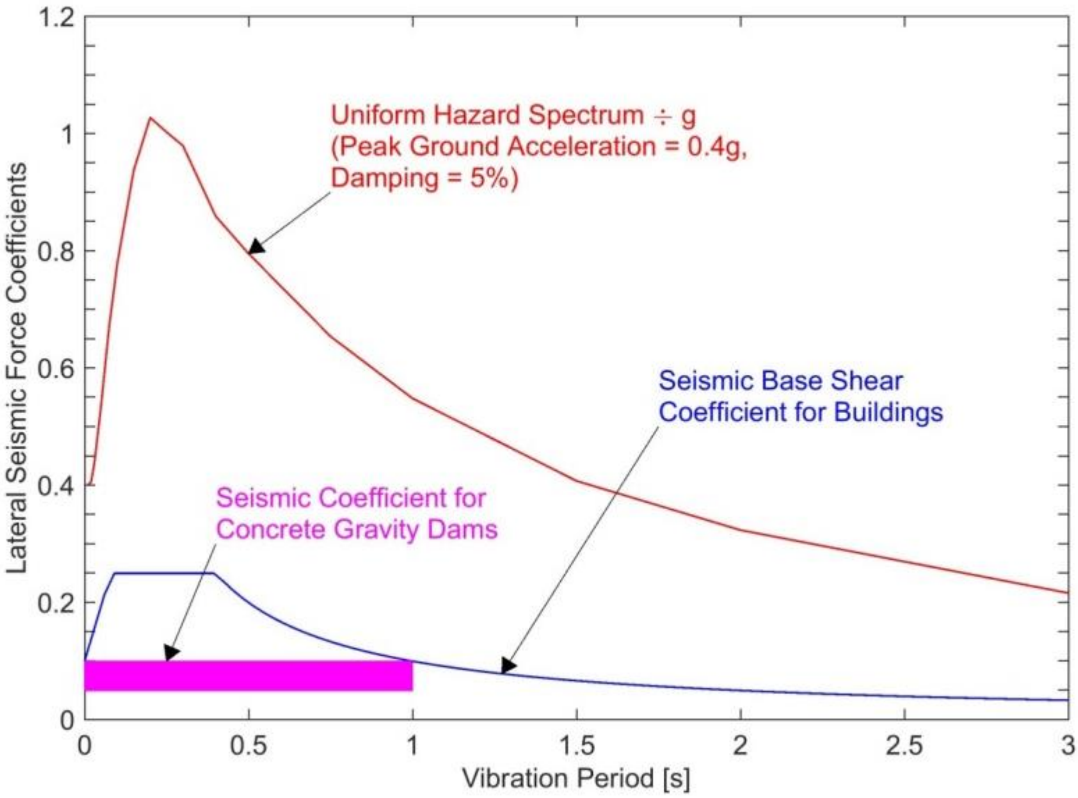

Figure 6-14: Comparison of uniform hazard spectrum and seismic coefficient for concrete dams and buildings (adapted from Chopra [1978]).

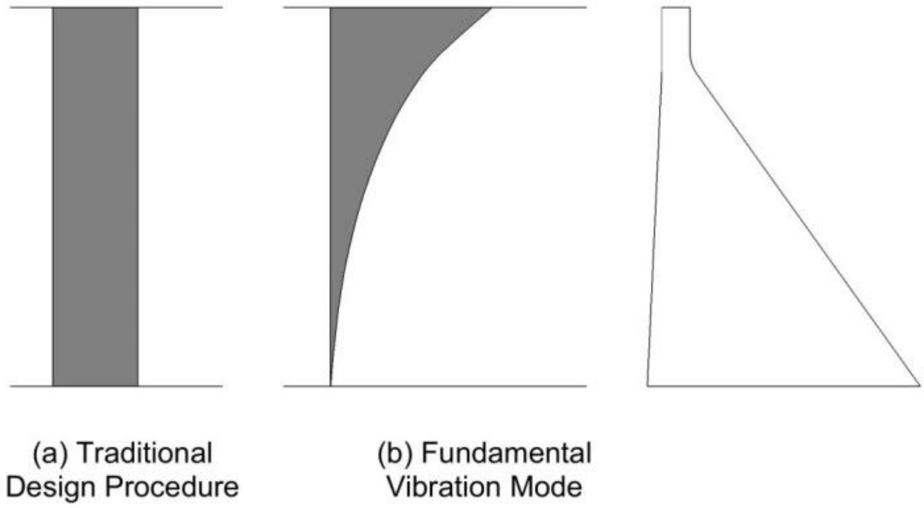

Figure 6-15: Distribution of seismic coefficients over dam height in traditional design and for the fundamental vibration mode (adapted from Chopra [1978]).

On the other hand, earthquake-induced stresses in gravity dams can be much larger—around 50%—when dam–water interaction arising from deformations of the dam and water compressibility effects are considered [Chopra et al., 1980; Fenves and Chopra, 1985b]. It is apparent, therefore, that hydrodynamic effects are considerably underestimated because of assumptions implicit in traditional design forces.

Finally, the static overturning and sliding criteria that have been used in traditional gravity dam design procedures have little meaning in the context of oscillatory response to earthquake motions.

6.11.4 Unrealistic Estimation of Seismic Demand and Structural Capacity

Traditional design procedures greatly underestimate seismic demands imposed on both arch and gravity dams, as well as the capacity of these structures to resist these demands. The seismic forces associated with the mass of the dam and the hydrodynamic pressures are underestimated, as mentioned earlier. The tensile strength of concrete, which is not insignificant, is essentially ignored in the no-tension requirement in the design criteria for gravity dams.

Methods for designing dams must be improved in at least two major ways: (1) seismic demands should be computed by dynamic response analysis of the dam–water–foundation system; and (2) the tensile strength of concrete should be determined by testing cylindrical cores that are large enough—diameter equal to 3 or 4 times the size of the coarse aggregate.

6.12 Dynamic Analysis Procedures

It is apparent from the preceding section that traditional seismic coefficient methods must be abandoned in favor of dynamic analysis procedures in order to reliably predict the earthquake-induced demands on dams. Various such procedures are mentioned in this section, together with their potential and limitations.

6.12.1 Reasons Why Standard Finite Element Method Is Inadequate

Because of the versatility of the finite element method (FEM) in modeling arbitrary geometries and variations of material properties, this method is suited for formulating a computational model of a concrete dam. In fact, analysis of the dam alone (no impounded water) supported on rigid foundation rock to ground motion specified at the base would be a standard application of the FEM. However, analysis of concrete dams is greatly complicated by the fact that the structure interacts with the water impounded in the reservoir and with the deformable foundation rock that supports it, and because the fluid and foundation domains extend to large distances.

The interaction mechanisms may be included in a crude way by combining finite element models for a limited extent of the impounded water and of the foundation rock with a finite element model of the dam, thus reducing the “semi-unbounded” system to a finite-sized model with rigid boundaries, which, generally, do not exist at the site (Figure-6-16). Such a model does not allow for radiation of hydrodynamic pressure waves in the upstream direction or stress waves in the foundation rock because these waves are reflected back from the rigid boundaries, thus trapping the energy in the bounded system. Thus, a significant energy loss mechanism, referred to as radiation damping, is not represented in the bounded models of the impounded water and foundation rock.

While research on modeling of the semi-unbounded geometry of the impounded water and foundation rock domains was in progress, an expedient solution was proposed by Clough [1980] that included in the finite element model a limited extent of foundation rock, assumed to have no mass, and modeled hydrodynamic effects by an added mass of water moving with the dam; the design ground motion defined typically at the ground surface was applied at the bottom fixed boundary of the foundation domain; see Figure 6-17. This modeling approach became popular in actual projects because it was easy to implement in commercial finite element software. However, such a model solves a problem that is very different from the real problem on two counts: (a) the assumptions of massless rock and incompressible water—implied by the added mass water model—are unrealistic, as will be demonstrated in Sections 6.12.3.3 and 6.12.3.4; and (2) applying ground motion specified at the ground surface to the bottom boundary of the finite element model contradicts the recorded evidence that motions at depth may differ significantly from surface motions.

6.12.2 Rigorous Methods

Earthquake analysis of dams should include the following factors: (1) the semi-

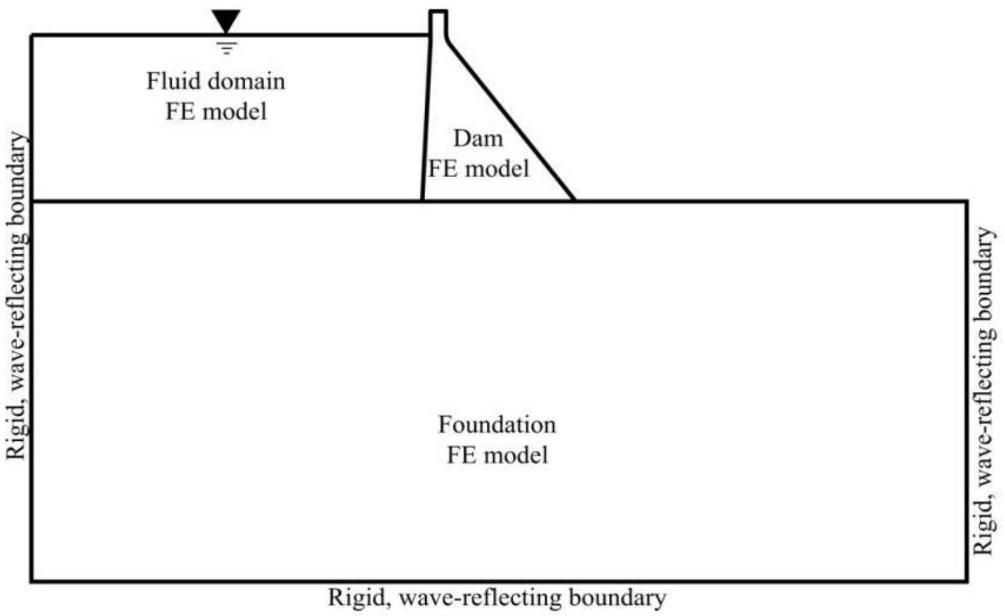

Figure 6-16: Standard finite element analysis model with rigid, wave-reflecting boundaries.

unbounded extent of the impounded water and foundation rock domains; (2) dam–foundation interaction considering mass, flexibility, and damping of rock; and (3) dam–water interaction considering compressibility of water. Two approaches exist for such rigorous analyses: the *substructure method* and a *direct finite element method*. Limited to analysis of linear systems, the substructure method is summarized in Section 6.12.3. A simplified version of the substructure method intended for preliminary analysis, design, and evaluation of gravity dams appears in Section 6.12.4. The Direct Finite Element Method (FEM) is summarized in Section 6.12.5.

6.12.3 Substructure Method: Response History Analysis of Linear Systems

6.12.3.1 Overall Concept

A substructure method to determine the earthquake response of concrete gravity dams as a function of time, including all the significant effects of dam–water–foundation interaction and sedimentary deposits at the reservoir bottom was developed by Fenves and Chopra [1984a]. This method determines the response of idealized systems shown in Figure 6-18 to free-field ground motion specified at the interface between the dam and foundation rock; this is the motion that would have existed in the absence of the dam and impounded water. The substructure method permits different types of models for the three substructures—dam, fluid domain, and foundation domain: finite element model for the dam; continuum model for the fluid domain unbounded in the upstream direction; and a viscoelastic half-space continuum model for the foundation domain of semi-unbounded geometry (Figure 6-18) without truncating these domains to finite size.

The substructure method is formulated in the frequency domain to determine the complex-valued frequency response functions, followed by Fourier synthesis of the responses to individual harmonic components to determine the responses—displacements and stresses—of the dam to free-field ground motion specified at the dam–foundation interface. The substructure method cannot be implemented in commercial finite element codes.

6.12.3.2 EAGD-84 Computer Program

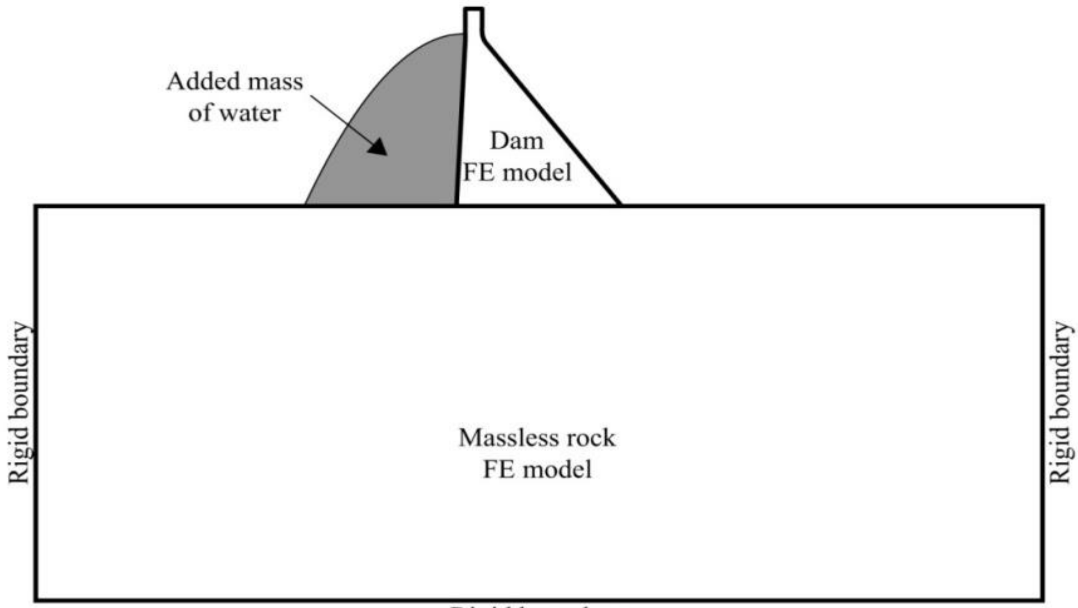

Figure 6-17: A popular finite element model that assumed foundation rock to have no mass and models hydrodynamic effects by an added mass of water moving with the dam.

The substructure method for earthquake analysis of gravity dams has been coded in the computer program EAGD-84 [Fenves and Chopra 1984b]. In this report the development of an appropriate idealization of the system is discussed, the required input data to the computer program are described, the output is explained, and the response results from a sample analysis are presented.

Two enhancements of the EAGD-84 program implemented recently should facilitate use of the program and expand the range of its applicability [Lokke and Chopra, 2013]. MATLAB modules were developed to facilitate development of the finite element model for the dam and to prepare data to be input into the program. Secondly, compliance data necessary to construct the dynamic stiffness matrix for the foundation were expanded. The original version of the program included such data for five values of the constant hysteretic damping factor η_f = 0.01, 0.10, 0.25, and 0.50, which in retrospect turned out to be too coarse. Compliance data have now been added for a closely-spaced set of η_f values.

Developed as a computer program for research purposes, EAGD-84 lacks the convenient user interfaces characteristic of commercial finite element codes; however, it has been used for many design and evaluation projects worldwide. It may be accessed from NISEE Library: http://nisee.berkeley.edu/elibrary/getpkg?id=EAGD84

6.12.3.3 Implications of Ignoring Water Incompressibility

Because dynamic analysis of dams is greatly simplified if compressibility of water is neglected, this assumption is attractive in engineering practice. However, it leads to erroneous results, as demonstrated next. Thus, modeling of hydrodynamic effects by an added mass of water moving with the dam, which neglects water incompressibility, is unacceptable.

Water compressibility plays an important role in the response of dams. In one example, neglecting water compressibility overestimated the stresses due to horizontal ground motion by 31–57% (Figure 6-19). In contrast, the response to vertical ground motion is underestimated by a factor of 5 if water compressibility is ignored; such underestimation is likely to occur in most cases. In some cases, the response to horizontal

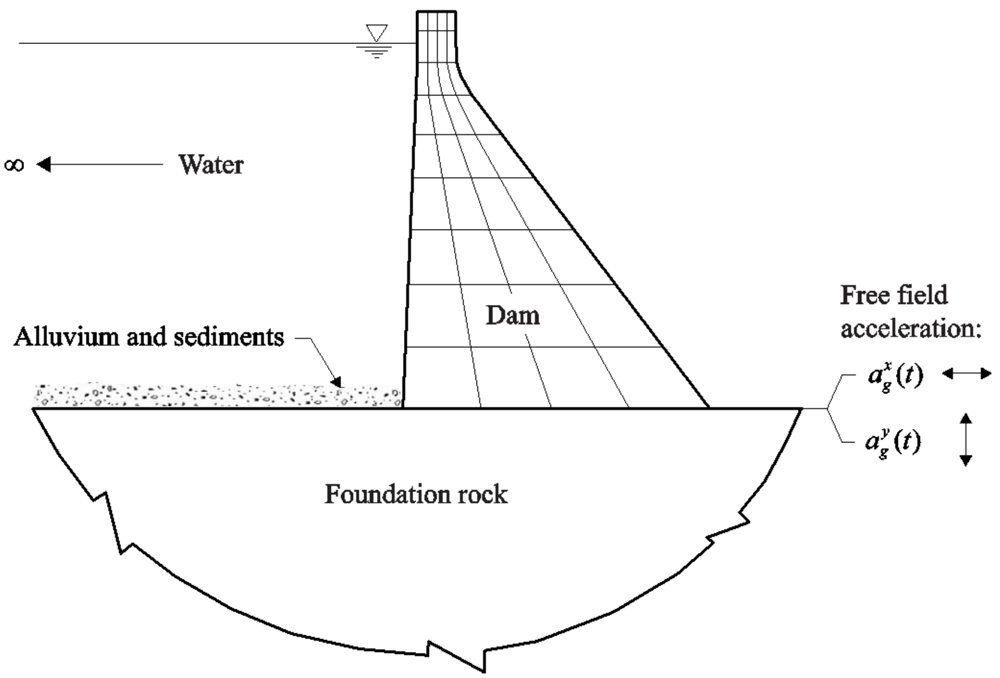

Figure 6-18 Dam-water-foundation system.

ground motion may be underestimated if water compressibility is neglected. For example, in one case the stresses were underestimated by 13–21%, depending on the location.

6.12.3.4 Implications of Ignoring Foundation Mass

The temptation to implement standard finite element analysis in commercial finite element software has motivated the practice of ignoring the mass of the foundation rock in earthquake analysis of dams. When rock is assumed to have no mass, radiation and material damping mechanisms characteristic of dam–foundation interaction do not develop, resulting in overestimation of stresses. In one example, by assuming foundation rock to be massless, the stresses were overestimated by 80% in parts of the dam (Figure 6-20). In many cases, such overestimation of stresses will lead to unnecessarily expensive designs for new dams, and to the erroneous conclusion than an existing dam is unsafe, thus requiring unnecessary retrofit that is invariable very expensive.

6.12.3.5 Water–Foundation Interaction

The substructure method does not lend itself to explicit modeling of water–foundation interaction. However, these interaction effects can be modeled indirectly by introducing a wave reflection coefficient, denoted by α, which in the substructure method should be computed from the properties of the underlying rock. Recent research has demonstrated that this simple model gives good results [Chopra, 2019; Appendix A5.2].

6.12.3.6 Sedimentary and Alluvial Deposits

The bottom of a reservoir upstream from a dam may consist of highly variable layers of exposed bed rock, alluvium, silt, and other sedimentary material. In the substructure method, the reservoir bottom is approximately modeled by a boundary that partially

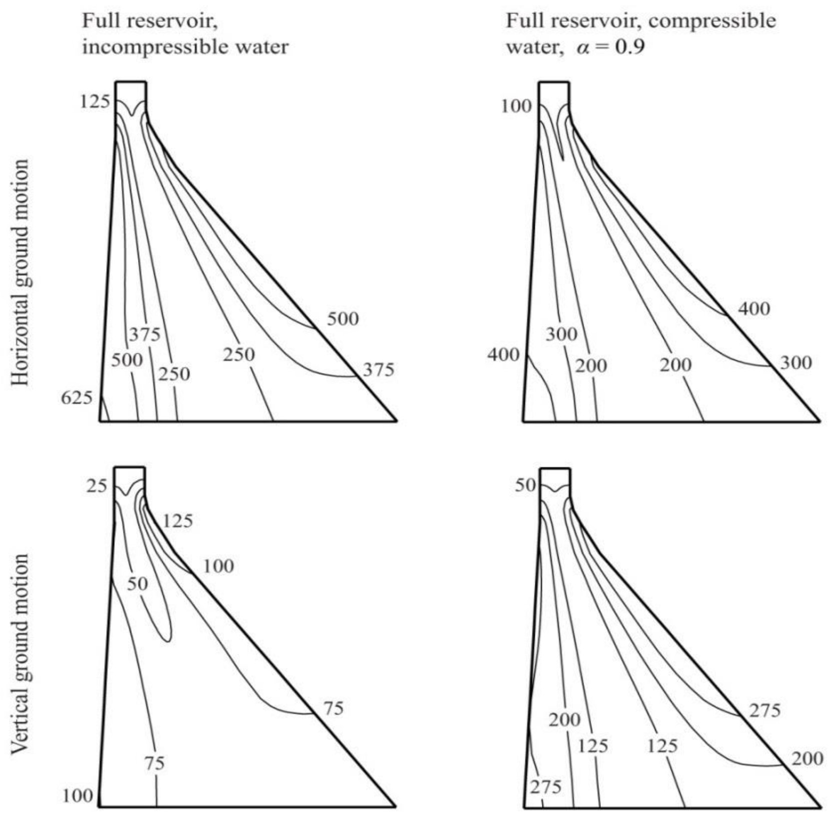

Figure 6-19: Influence of water compressibility on envelope values of maximum principal stresses, in psi, in Pine Flat Dam with = 4 million psi supported on rigid foundation due to horizontal and vertical components of Taft ground motion.

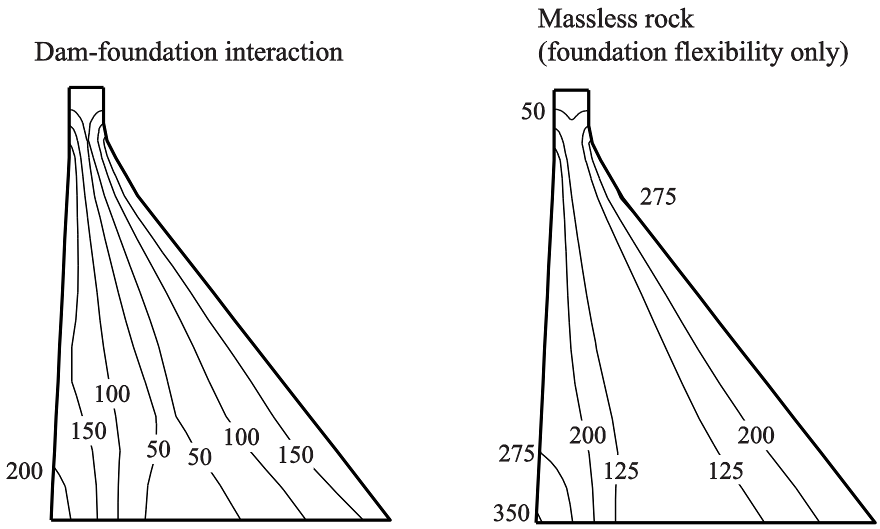

Figure 6-20:Influence of foundation idealization on envelope values of maximum principal stresses, in psi, in Pine Flat Dam due to horizontal ground motion. Results are presented for two cases: (1) including all effects of dam–foundation interaction; (2) and assuming rock to be massless, i.e., considering foundation flexibility only.

absorbs incident hydrodynamic pressure waves. Original research on this topic had concluded that the response of gravity dams varies considerably with α [Fenves and Chopra, 1983]. However, recent research has demonstrated that when water–foundation interaction effects are included in the substructure method, sediments may be ignored in the analysis [Chopra, 2019; Section 11.10.5].

6.12.4 Response Spectrum Analysis of Non-Overflow Monoliths

6.12.4.1 Overall Concept

The response spectrum analysis (RSA) procedure [Chopra, 1978; Fenves and Chopra, 1985, Fenves and Chopra, 1987; Lokke and Chopra, 2013; and Lokke and Chopra, 2015] to estimate the earthquake-induced stresses in concrete gravity dams considers only the more significant aspects of the response. Although the dynamics of the system including dam–water–foundation interaction is considered in estimating the response due to the fundamental vibration mode, the less significant part of the response due to higher modes is estimated by the static correction method. Only the horizontal component of ground motion is considered because the response due to the vertical component is known to be much smaller. This procedure is included as Appendix C in this Manual.

Dam−water−foundation interaction introduces frequency-dependent, complex-valued hydrodynamic and foundation terms in the governing equations. Based on a series of approximations, frequency-independent values of these terms were defined and an equivalent SDF system developed to estimate the fundamental mode response of dams. These concepts lead to the RSA procedure presented in Appendix C. Recognizing that the cross-sectional geometry of concrete gravity dams does not vary widely, standard data for the vibrational properties of dams and for parameters that characterize dam−water interaction (including reservoir bottom absorption) and dam−foundation interaction are presented to facilitate implementation of the procedure.

6.12.4.2 CADAM Computer Program

CADAM—computer-aided stability analysis of gravity dams—a program developed at the École Polytechnique de Montreal, Canada [Leclerc et al. 2003], implements the response spectrum analysis procedure

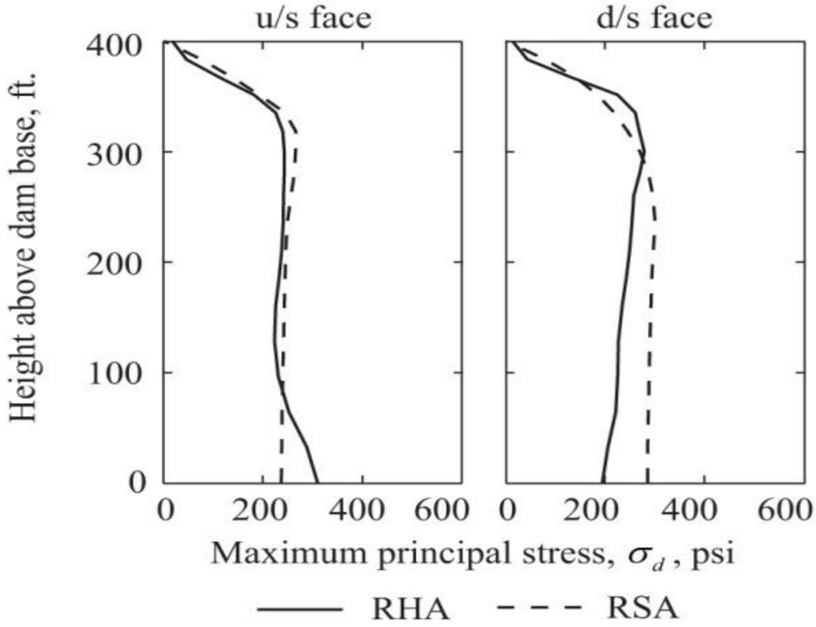

Figure 6-21: Comparison of peak values of maximum principal stresses at the upstream and downstream faces of a dam supported on a flexible foundation with full reservoir computed by RSA and RHA procedures; initial static stresses are excluded.

described in the preceding section. This procedure is referred to as the "pseudo-dynamic method" in CADAM, which also includes several other analysis options. An object-oriented program that offers a versatile computing environment, CADAM is convenient to use. It can be downloaded from http://www.polymtl.ca/structures/en/telecharg/cadam/telechargement.php, where a Users' Manual is also available.

6.12.4.3 Accuracy of Response Spectrum Analysis

Presented in Appendix D is a comprehensive evaluation of the accuracy of the RSA procedure by comparing its results with those obtained from response history analysis (RHA) of the dam modeled as a finite element system, including dam–water–foundation interaction by the substructure method (Section 6.12.3). Comparison of the median of the peak responses of an actual dam to 58 ground motions determined by both procedures demonstrates that the RSA procedure estimates stresses to a degree of accuracy that is satisfactory for the preliminary phase in the design of new dams and in the safety evaluation of existing dams. A representative Figure 6-21 demonstrates that the RSA procedure provides very good estimates of the principal stresses.

6.12.5 Response Spectrum Analysis of Gated Spillway Monoliths

The RSA procedure for non-overflow monoliths described in Section 6.12.4 is also applicable to gated spillway monoliths. Because the cross-sectional geometry is now different, standard data for the vibrational properties of monoliths and for parameters that characterize dam–water interaction (including reservoir bottom absorption), and dam–foundation interaction were developed specifically for gated spillway monoliths [Chopra and Tan, 1989]. This report is included as Appendix E+ in this Manual.

+ **In using this Appendix E, it should be recognized that parameters that characterize dam–foundation interaction are available for fewer values of the foundation damping factor η_f compared to Appendix D for non-overflow monoliths where data are presented for a set of closely-**

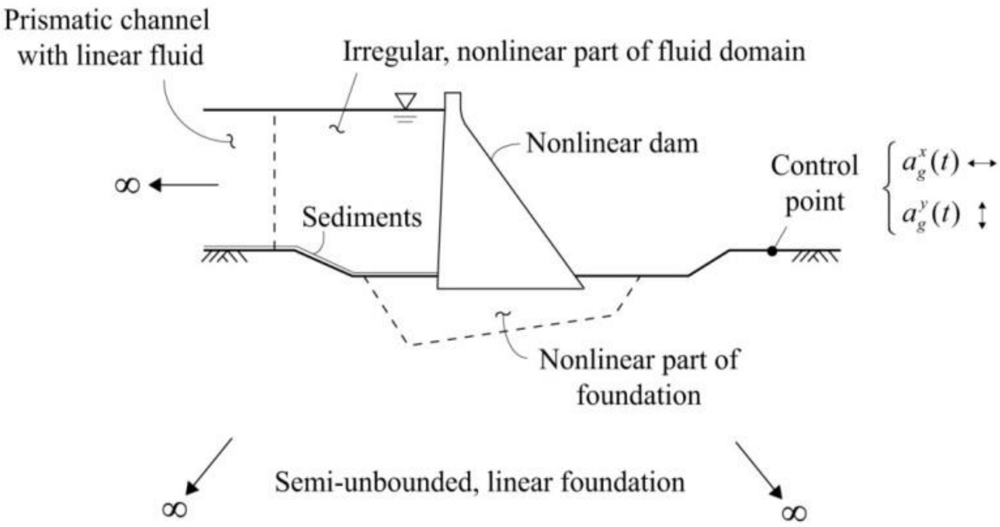

Figure 6-22: Semi-unbounded dam–water–foundation rock system showing main components: (1) the dam itself; (2) the foundation rock, consisting of a bounded, nonlinear region and a semi-unbounded, linear region; and (3) the fluid domain, consisting of an irregular, nonlinear region and a semi-unbounded prismatic channel with linear fluid.

6.12.6 Direct Finite-Element Method: Linear and Nonlinear RHA

Although linear analyses have provided great insight into the earthquake response of concrete dams, it is evident that a reliable estimate of the seismic safety of a dam can be obtained only by a nonlinear analysis if the earthquake damage is expected to be significant. The response of the nonlinear system shown in Figure 6-22 with semi-unbounded geometry of the foundation and fluid domains—including dam–water–foundation interaction is to be determined. The Direct FEM [Lokke and Chopra 2017; 2018; 2019] introduces wave-absorbing (or wave-transmitting) boundaries at two locations: (1) upstream end of the fluid domain to model its essentially infinite length; and (2) the bottom and side boundaries of the foundation domain to model its semi-unbounded geometry (Figure 6-23). The finite-element model of the fluid domain now includes water compressibility, and the finite element model of the foundation domain includes mass, stiffness, and material damping appropriate for the rock; water–foundation interaction is also included. Thus, the untenable assumptions of massless rock and incompressible water in the popular FEM (Section 6.12.1) are eliminated. An example of such a model is shown in Figure 6-24.

The earthquake excitation also is more realistically defined in the Direct FEM compared to the popular FEM. The excitation defined at the bottom and side boundaries of the foundation domain is determined by deconvolution of the design ground motion, typically specified on level ground at the elevation of the abutments in a two-dimensional model, it may be specified near the base of the dam (Figure 6-22). The resulting spatially varying motions cannot be input directly at wave-transmitting boundaries; instead, tractions determined from these motions are converted to effective earthquake forces.

spaced values of η_f. However, this limitation can be overcome by linearly interpolating between the two nearest values of η_f for which data are available. Furthermore, as recommended in Appendix D, stresses on the sloping part of the downstream face computed by beam theory should be multiplied by a correction factor of 0.75.

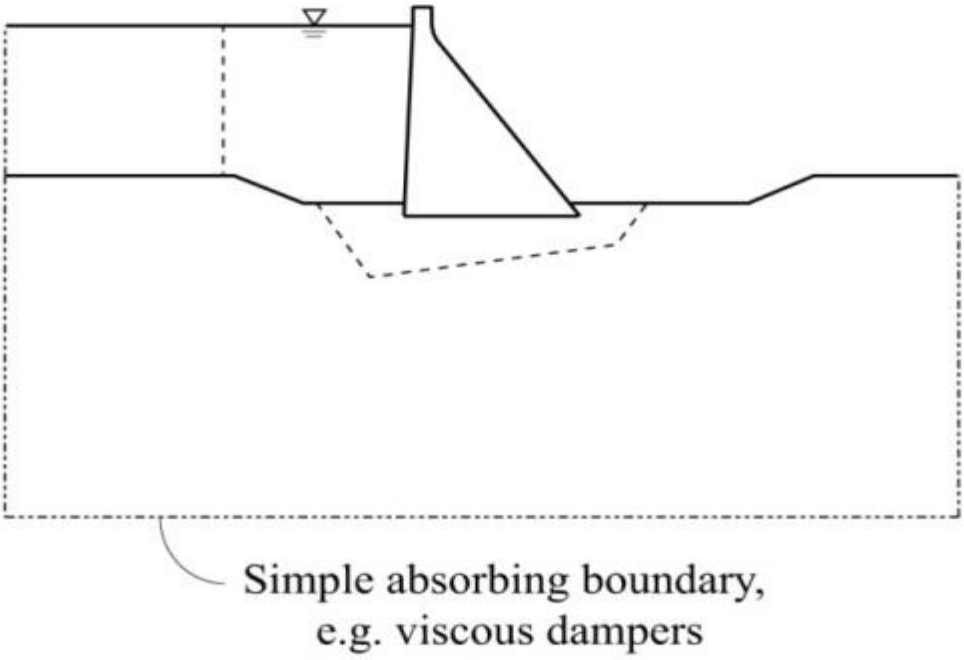

Figure 6-24: Dam–water–foundation system with truncated foundation and fluid domains and wave-absorbing boundaries.

The Direct FEM has the great advantage over the substructure method in that is it applicable to nonlinear systems, thus permitting modeling of concrete cracking, as well as sliding and separation at contraction joints, lift joints, dam–foundation interface, and fissures in rocks; however, it has the disadvantage in that it requires truncation of fluid and foundation domains, thus requiring absorbing boundaries to simulate their semi-unbounded size. The procedure presented in Lokke and Chopra [2017; 2018] can be implemented in almost every commercial FE code without requiring modification of the source code. To achieve this goal, viscous dampers were selected to model wave-absorption boundaries to simulate the semi-

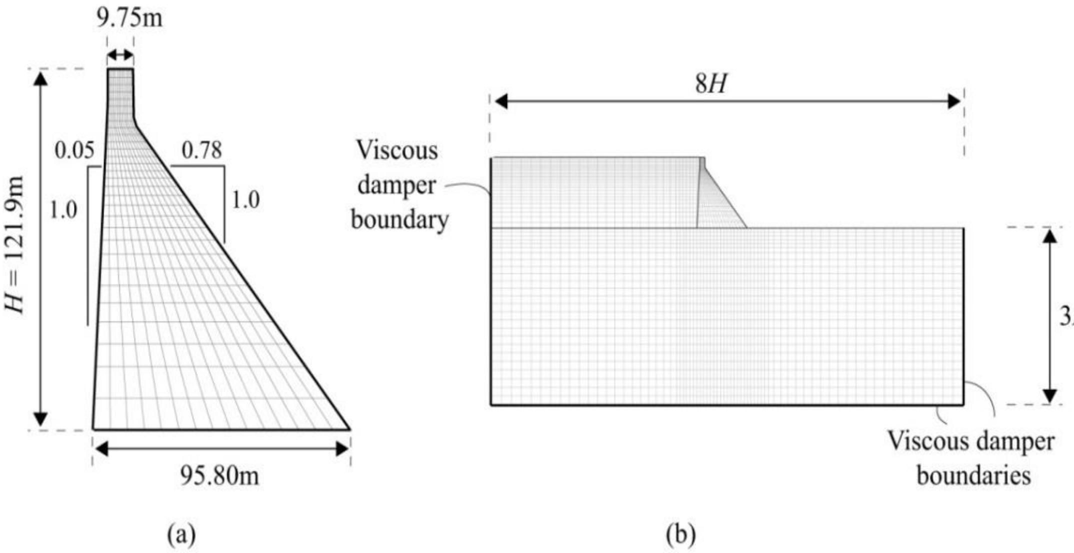

Figure 6-23: Finite-element model for dam–water-foundation system with truncated foundation and fluid domains and wave-absorbing boundaries.

unbounded domains and a theory developed to compute (in an auxiliary analysis) the effective earthquake forces that are applied at these boundaries.

The Direct FEM has been validated against the completely independent structure method and shown to produce essentially identical results [Lokke and Chopra 2017; 2018]. This validation confirms that (1) the wave-absorbing boundaries are effective in simulating the semi-unbounded geometry of fluid and foundation domains; and (2) the effective earthquake forces represent properly the earthquake excitation.

6.12.7 Calibration of Numerical Model: Damping

The numerical model for a concrete dam should be calibrated to match its actual vibration properties. Although the need to match vibration frequencies and modes is widely recognized, calibration of damping has not received as much attention.

Damping in the numerical model for the dam–water–foundation system should be consistent with measured values determined from low-amplitude motions—within the linear range of response—recorded during forced vibration tests, ambient vibration, or small earthquakes. Obviously, the measured values represent the overall damping in the system, including material damping, radiation ramping, and energy loss at reservoir boundaries; information on the contributions of individual sources of damping is generally not available.

Summarized in Figure 6-25 is the data for damping "measured" at thirty-two concrete dams determined by forced vibration tests and ambient vibration measurements [Chopra 2019; Section 10.3]. Both gravity dams and arch dams covering a wide range of system parameters are included. The overall damping values measured at these dams are, but for a few exceptions, all in the range of 1%–5%. These comprehensive data lead to an important conclusion: overall damping in the numerical model should not exceed 5% unless a larger value was "measured" at the particular dam. In contrast, current practice of specifying a viscous damping ratio of 5% for the concrete dam alone and a similar value for the foundation domain separately will lead to damping in the range of 10–20% in the overall dam–water–foundation- system. Thus, the current practice of choosing damping values should be abandoned because it will significantly underestimate the earthquake response of dams.

Researchers have demonstrated that damping in the range of 1–2% for the dam and 1–4% for the foundation is likely to lead to an overall damping in three-dimensional nu-

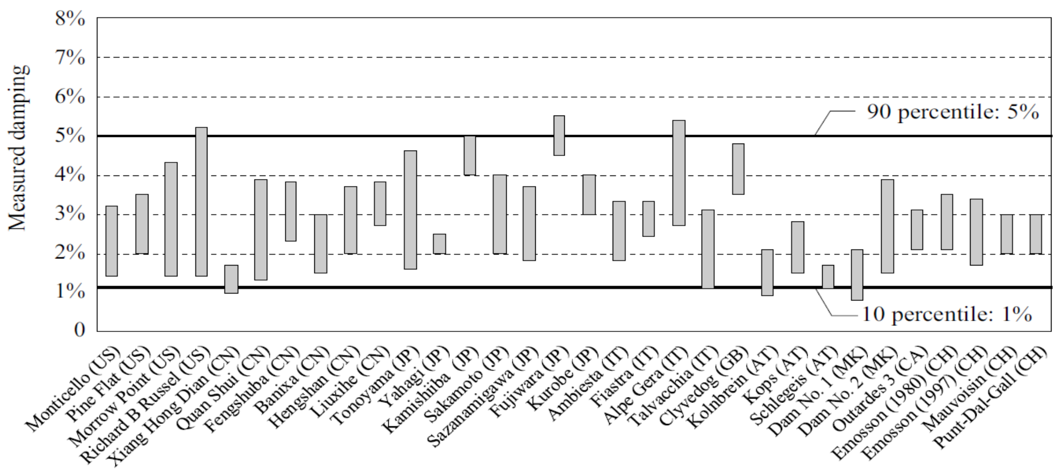

Figure 6-25: Measured damping at 32 concrete dams during forced vibration measurements compiled from Hall [1988], Proulx and Paultre [1997], and Proulx et al. [2004]. The range for each dam shows the minimum and maximum damping values measured in the first few (1 to 5) resonant frequencies.

merical models that is consistent with measured values. However, limiting the overall damping to less than 5% in 2D numerical models is very difficult because of the large amount of radiation damping associated with two-dimensional homogeneous, semi-unbounded foundation models [Chopra 2019; Section 10.3].

Overall damping in a two-dimensional model of a gravity dam is estimated as an intermediate step in response spectrum analyses (Appendices D and E). Any value larger than 5% is not permitted for seismic design or safety evaluation unless a larger value was "measured" at the particular dam.

6.13 Performance Evaluation

6.13.1 Progressive Seismic Demand Analyses

The seismic demands imposed on the dam should be determined by a series of dynamic analyses that become progressively more rigorous. Each of these analyses should include the effects of dam–water–foundation interaction and the semi-unbounded extent of foundation and fluid domains, demonstrated to be significant in earlier sections.

The seismic demands imposed by the OBE can be computed by linear analysis in two stages: (1) a simplified RSA in which the response is estimated directly from the earthquake design spectrum, considering only those factors that are most important in the earthquake response of dams and yet is simple enough not to require the use of elaborate commercial computer programs (Sections 6.12.4 and 6.12.5); and (2) a refined RHA procedure for finite-element idealization of the dam including dam–water–foundation interaction (Sections 6.12.3 and 6.12.6). The former is recommended for purposes of preliminary design, and the latter for accurate computation of the dynamic response necessary to check the adequacy of the structure designed for the preliminary design forces. As mentioned earlier, computer programs are available to implement refined RHA procedures for concrete dams.

The design should provide against overstressing in compression and tension; that is, the compressive and tensile stresses should not exceed the compressive and tensile strengths of concrete, respectively. The concrete strength requirements will be controlled by the tensile stresses because they will be similar in magnitude to the compressive stresses, whereas the tensile strength of mass concrete is an order of magnitude smaller than the compressive strength. The overturning and sliding stability criteria that have been used in standard design procedures in the past have little meaning in the context of oscillatory response of dams due to earthquakes [Chopra and Zhang, 1991]. These criteria could be satisfied only because the lateral earthquake force was unrealistically small in traditional design (Sections 6.11.1 to 6.11.3). However, they cannot be satisfied if the peak lateral force is determined by dynamic analyses. Researchers have proposed reducing this force to 50–60% of its full value in stability analysis of the dam [Tinawi et al., 2000]. The end result of this phase of the design process is a preliminary design of the dam.

The adequacy of the preliminary design of the dam should be checked with the aid of refined, rigorous analysis procedures, such as those mentioned in Section 6.12.3 and 6.12.6. The response of the preliminary design of the dam to selected ground motions should be determined, resulting in more accurate values for the stresses and internal forces. Based on these results, the preliminary design of the dam should be revised, if necessary, to satisfy the same design criteria as mentioned in this section. The design modification may involve reshaping the cross section, increasing the thickness of a gravity dam, and/or increasing the concrete strength.

However, the stresses in gravity dams can be significantly reduced by modifying the usual designs to reduce the weight near the crest of the dam. Instead of the solid concrete block added near the crest in typical designs of dams to support the roadway and to resist the impact of floating objects, lightweight structural systems would be preferable [Chopra, 2019, Section 7.2.4]. Similarly, the auxiliary structures usually appended on the top of dams should be located with discretion so that they have a minimum adverse effect on stresses in the dam. Possible modifications in the geometry and mass distribution of arch dams that might lead to reduction of earthquake-induced stresses remain to be investigated.

A dam designed to remain within the linear range of behavior during the OBE should be evaluated to determine its performance in the event of a MDE. Before embarking upon a nonlinear RHA accompanied by a multitude of challenges in developing a numerical model, defining nonlinear constitutive properties of the materials, and dealing with sensitivity of results to uncertainty in ground motion and material properties, the most rigorous linear RHA should be implemented. The results of such linear analysis would provide an initial understanding of degree of nonlinearity to be expected during the MDE. Such results can also assist in identifying areas of the dam that are likely to be strained beyond the linear range and require carefully developed nonlinear models. If the results of linear analysis indicate that the tensile strength of concrete or a joint is exceeded repeatedly during the duration of shaking, the designer should consider the possibility of modifying the design to ensure essentially linear response even during the MDE. This could very well be economically preferable over repairing the damage that the original design is expected to experience during an MDE.

Many of the preceding comments in this section, after obvious modification, carry over to seismic evaluation of existing dams. In particular, a rigorous linear RHA should still be the first step in computing seismic demands on the dam, and the same criteria should be employed to determine the need for nonlinear RHA. In past investigations (of actual projects) that ignored mass of foundation rock and compressibility of water, the seismic demands on the dam were overestimated by factors up to 2 or 3 [Chopra, 2012]. Such results could have led to the erroneous conclusion that an existing dam is unsafe, thus requiring upgrading, which is invariably very expensive.

However, if rigorous linear RHA of the dam demonstrates the potential for damage, a nonlinear analysis would be required. Despite its aforementioned limitations, nonlinear RHA of the dam–water–foundation system will provide a rough estimate of cracking in concrete, the amount of opening and sliding at contraction joints, lift lines, and joints in the rock. If this damage is deemed to be unacceptable, a retrofit scheme for the dam should be designed and earthquake response of the retrofitted dam computed to ensure that it meets the performance criteria.

6.13.2 Progressive Capacity Evaluation

An important property that determines the capacity of concrete dams to withstand earthquakes is the tensile strength of concrete. Ideally, the tensile strength should be determined from appropriate tests on specimens of concrete for the particular dam. However, a preliminary estimate of the tensile strength can be obtained from Figure 6-26 [Raphael, 1984], which presents four

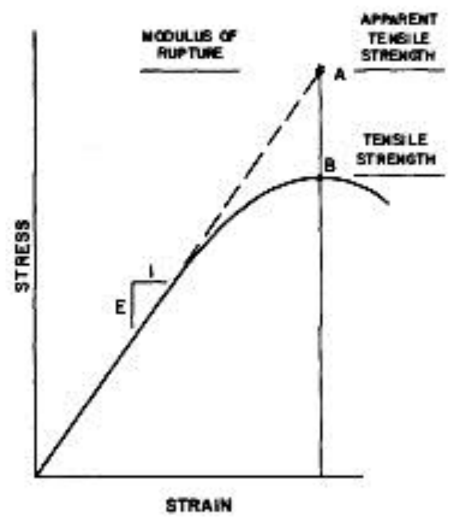

Figure 6-26: Apparent tensile strength [Raphael, 1984].

plots of tensile strength as a function of compressive strength, to be used depending on application. The lowest two plots, $f_t = 1.7 f_c^{2/3}$ and $f_t = 2.3 f_c^{2/3}$, are for long-time or static loading. The lowest curve represents actual tensile strength, whereas the second represents “apparent” tensile strength. The latter is not a quantity that can be measured; it is simply the stress corresponding to tensile strain at failure under the assumption of a linear stress–strain curve; see Figure 6-27. The apparent tensile strength is to be used to interpret the stresses computed by linear finite-element analysis. Similarly, the third and fourth plots, $f_t = 2.6 f_c^{2/3}$ and $f_t = 3.4 f_c^{2/3}$, are the actual and “apparent” tensile strengths at strain rates expected during earthquake-induced vibration.

If the stresses computed from linear RHA exceed repeatedly the “apparent” tensile strength determined by the empirical methods mentioned earlier, this important property should be determined for the actual concrete in the dam. Tensile strength can be determined from three types of tests: direct tension, splitting tension, and flexural tests. Results of these tests differ, and results of tests on cores taken in the field differ compared to tests on laboratory specimens. The direct tension test is difficult to accomplish and underestimates the tensile strength of concrete if the specimen is allowed to surface-dry. The flexural test, together with its usual linearly derived modulus of rupture, provides a basis to determine the tensile strength. The modulus of rupture should be multiplied by a factor that accounts for the nonlinear behavior of concrete and depends on the shape of the specimen. On the other hand, the splitting tension test is easiest to accomplish and gives the most reliable results. However, tensile strength obtained from the splitting tension test should be multiplied by about 4/3 to account for the nonlinear behavior of concrete near failure, before using it to interpret results of linear finite-element analysis [Raphael 1984].

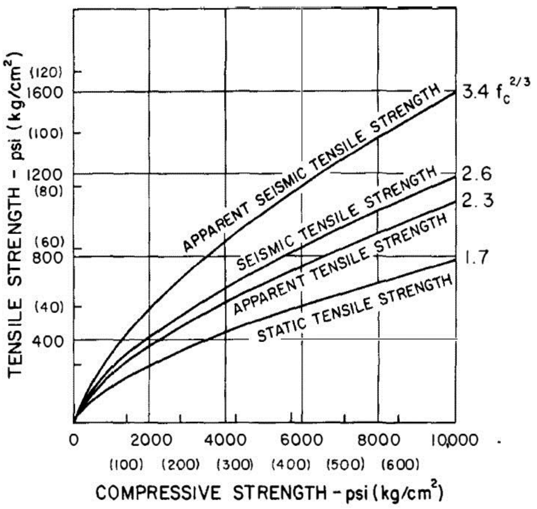

Figure 6-27: Design chart for tensile strength [Raphael 1984]

Because the tensile strength of concrete increases with the rate of loading the aforementioned tests should be conducted at strain rates the concrete may experience during earthquake motions of the dam. In the absence of a facility to perform dynamic tests. Raphael [1984] recommended that the tensile strength of concrete for judging the seismic safety of a concrete dam be equal to the static value multiplied by 1.5. However, test data on concrete for some dams does not support this recommendation, nor is it appropriate in the presence of significant initial (static) tensile stresses in parts of arch dams. Thus, a smaller value, say, 1.25, is recommended unless evidence is available to justify a larger value.

These estimates of tensile strength are appropriate for mass concrete but not for weaker zones, e.g., horizontal lift joints in all types of dams and vertical contraction joints between cantilevers of arch dams. The tensile capacity of these joints is greatly influenced by the construction methods and details.

6.13.3 Evaluating Seismic Performance

Evaluating the seismic performance of concrete dams based on results of linear RHA is relatively straightforward. If computed tensile stresses do not exceed the tensile strength—which may be reduced by a factor to be conservative—we conclude that the dam will remain undamaged during the earthquake. As mentioned earlier, typically this is the performance requirement during the OBE.

Evaluating the seismic performance of dams subjected to ground motions intense enough to cause damage is very challenging. Quantitative measures for the extent of damage—cracking in concrete, sliding at lift joints or at cracked interfaces, and opening of contraction joints—that dams can sustain and still retain the impounded water have not been developed for lack of research on sensitivity of computed response to uncertainty in material properties and GMs, and on experimental validation of results from dynamic analysis. Thus, performance evaluation of dams deforming beyond the linear range of behavior is open to interpretation and judgment, leaving open the possibility of different engineers arriving at contradictory conclusions.

After completing a nonlinear RHA of the dam, a post-earthquake analysis of the damaged dam is required to evaluate if the dam will remain stable and continue to contain the impounded water. Such analysis should model the dam in its damaged condition with uplift pressures modified to reflect the post-earthquake condition of the drains. Comprehensive but qualitative discussion of these topics is available in Chapter 6 of a National Research Council Report [1990].

Part of the difficulty in establishing quantitative criteria for evaluating results from nonlinear RHA is due to the dearth of definitive evidence—experimental or observational—on the evolution of failure mechanisms in concrete dams. There is a crying need for research on credible potential failure modes and how they could develop during an earthquake.

Evaluating foundation stability is also a very challenging problem. Results of nonlinear RHA by the direct FEM (Section 6.12.6) provide time variation of forces acting on the foundation. Under these driving forces, the dam should remain stable against sliding along concrete-rock contact and the foundation blocks or wedges formed by intersecting rock discontinuities should also remain stable. Evaluating the performance of the dam and foundation rock against these criteria is challenging, especially because the driving forces vary with time. This is yet another subject where much research is necessary to develop methodologies and to demonstrate their reliability.

The need for research alluded to in this section was articulated almost thirty years ago by a panel of experts appointed by the National Research Council [1990]. Progress since then has been meagre for lack of research funding.

6.13.4 Potential Failure Mode Analysis

It is useful to think of the various modes by which a dam can fail during an earthquake; failure is defined as uncontrolled release of the impounded water. Several potential failure modes could be identified, which in the context of gravity dams include (Figure-6-28).

- Concentration of stress in the upper part of the dam, where the slope of the downstream face changes, resulting in cracking extending through the thickness of the monolith and finally resulting in excessive movement of the separated block above the crack.

- Sliding along the dam–rock interface that may cause shearing of the vertical drains, making them ineffective, resulting in increased uplift pressures and reduced frictional force. Failure may result from excessive sliding at the interface or by toppling of spillway piers (about the weak axis) or slender blocks of arch dams.

- Sliding along an unbounded lift joint that may cause shearing of the vertical drains, making them ineffective, resulting in increased uplift pressures and reduced frictional force. Failure may result from excessive sliding along the lift joint or by toppling of spillway piers or slender blocks of arch dams.

- Sliding along a discontinuity in the foundation rock, resulting in excessive movement of the foundation blocks.

Some dam engineering organizations have now adopted a methodology for evaluating the seismic safety of existing dams or proposed new dams that consist of the following: (1) identify potential failure modes; (2) postulate the entire sequence of events leading to failure; and (3) develop logic (or event) trees, a graphical representation of all potential paths to failure. Documented in various publications [FEMA, 2014; Hartford and Baecher, 2004], this methodology is described is beyond the scope of this book. We simply observe that quantitative implementation of this methodology is beyond the scope of this Manual.

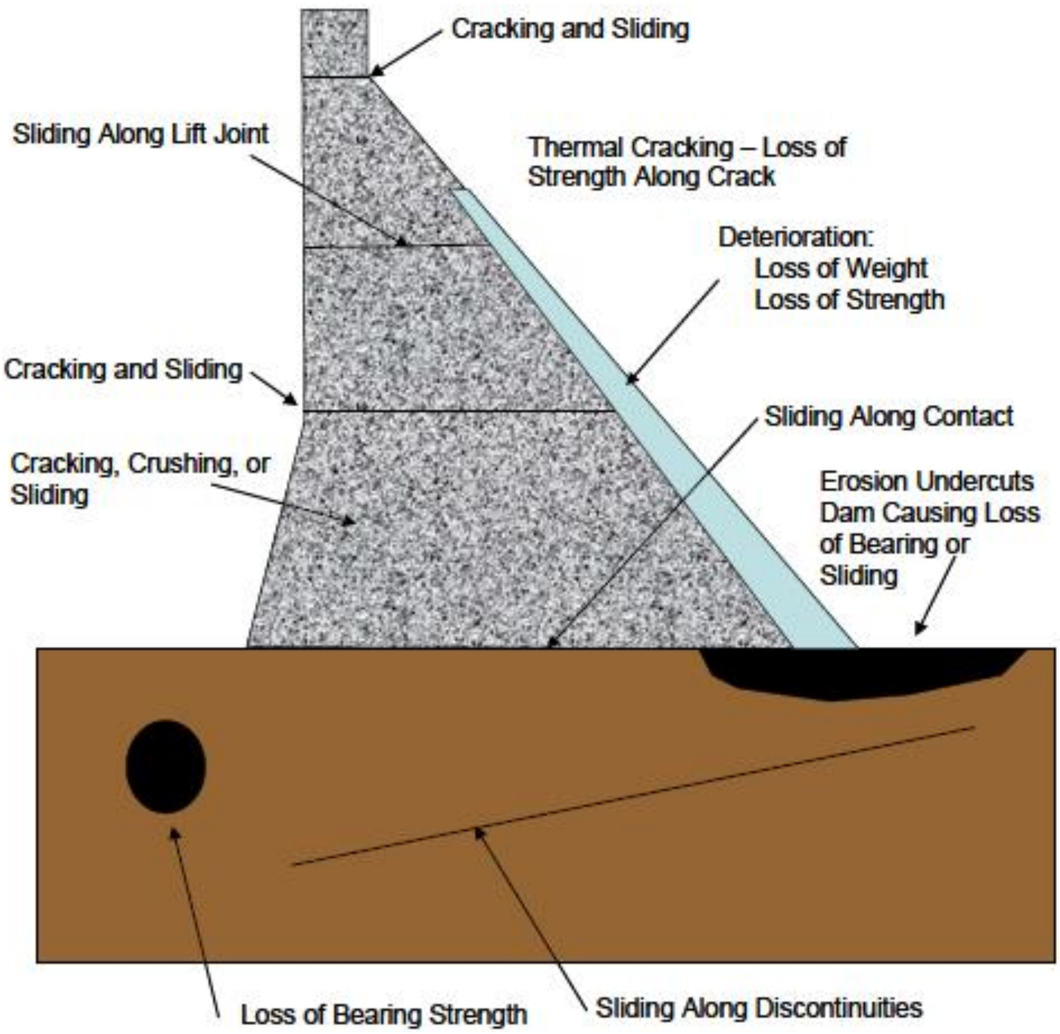

Figure 6-28**:** Potential failure modes given an initiating event [FEMA, 2014].

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Chapter 7. EARTH AND ROCK FILL DAMS

7.1 Introduction

An embankment dam is generally defined as one constructed of natural materials. The two principal types of embankment dams are earth and rock-fill dams, depending on the predominant fill material used. In India and elsewhere earth-fill dams are the most common, mainly because their construction uses materials locally available (including from excavations for other structures) with a minimum of processing and can be adapted to all types of foundations.

Earth fill dams have been constructed as both homogeneous or zoned dams (see Fig. 7-1 for typical sections). A homogeneous earth fill dam is composed of materials having essentially the same physical properties throughout the cross-section (except for filter, drain, and slope protection). The material comprising the dam must be sufficiently impervious to provide an adequate water barrier. Modern homogeneous dams usually incorporate some form of drainage zones for controlling internal pore water pressure and seepage forces; however, many small older dams do not have these provisions and must be inspected and monitored very carefully (e.g. Willington dam in Tamil Nadu, Mallaghata dam in Karnataka, etc.).

Zoned earth fill dams are usually constructed in areas where there is availability of several material types such as clays, silts, sands, gravels, and rock. A typical zoned earth-fill dam is composed of an impervious zone (or core or hurting) of fine-grained soils located within the interior of the cross-section and supported by outer zones (or shells or casing) of more pervious sand, gravel, cobbles, or rock fragments.

Impervious core rock-fill dams consist of an interior impervious zone or element supported by zones of compacted rock. The interior element controls the retention of the water and is usually a compacted impervious soil protected by filter and transition zones. The composition and construction of the filter and transition zones are especially critical.

Upstream faced rock-fill dams (concrete-faced, asphalt-faced, etc.) consist of a pervious rock embankment with an impermeable membrane on the upstream face. The rock mass provides stability and the membrane, or facing, retains the water. A special zone of selected small-size rock is used to support the face. The main body of the embankment is zoned with the rock sizes in the zones increasing toward the downstream face.

Embankment dams can be constructed on foundations that would be unsuitable for concrete dams. The foundation requirements for earth-fill dams are less stringent than those for rock-fill dams.

7.2 Structural Safety

The structural safety of an embankment dam depends on the absence of excessive deformations under all conditions of loading and operation, the ability to safely pass flood flows without overtopping the embankment, and the control of seepage to prevent piping of materials and to control pore pressures and thus prevent adverse effects on stability.

To properly evaluate the structural safety of an embankment dam, the following areas should be reviewed: embankment zoning and cross section; seepage control measures (drains and filters); predicted and measured deformation; erosion control measures such as rip rap bedding and filters; structural stability analyses; overtopping potential; foundation and embankment material properties and strengths; and adequacy of freeboard. For existing dams, the review should also include summarizing the past behavior of the dam, with attention given to any problem areas noted, changes in measured seepage, changes in measured pore pressures, changes in measured settlements and horizontal movements.

A number of studies have been made of dam failures and accidents. The results of one survey, by ICOLD were reported in its publication "Lessons from Dam Incidents (1979 Transactions of 13th ICOLD Congress, New Delhi)". The study shows that *foundation defects*, *overtopping* and *piping* are the three main causes of dam failures. For embankment dams, overtopping and piping have higher rate of incidence while for concrete dams, foundation defects have higher rates of failure incidence followed by overtopping.

From the study, it was also seen that foundation failures occurred relatively at early age of dams, while the other causes may take much longer to materialize. A study of the heights of the failed dams showed that 50% of the failed dams were between 15 and 20 meters high. A relation between dams built and failed for the various dam types from 1900 to 1969 indicated that concrete gravity dams appear the safest, followed by arch and embankment dams. Buttress dams have the poorest record but are also the ones used least. The study finally revealed improvement in design and lesser rate of failure over the 1900-1975 period and showed that modern embankment and concrete dams are both equally safe.

7.3 Dam Zoning

Embankment dams are generally designed to satisfy the particular topographic and foundation conditions at the site and to use available construction materials, so there really are no "typical" or "standard" designs. Good dam engineering involves use of the materials available near the site rather than to look for materials with preconceived ideas about the material properties needed. However, this rule is not followed in the search for critical filter and drain zones where invariably one might seek materials satisfying carefully specified particle size grading limits and dense hard durable materials. Table 7-1 below summarizes typical construction materials for different zones of embankment dams (Fell, et. al., 2005). IS 1498 & 8828 also provide useful guidelines for suitability of different materials for embankment dams.

7.4 Internal Erosion and Piping

As discussed above internal erosion and piping is among the major cause of embankment dam failure. Piping may occur in the embankment, foundation and embankment to foundation. For internal erosion and piping to occur four conditions must exist (Fell et. al., 2005):

- There must be a seepage flow path and a source of water;
- There must be erodible material within the flow path and this material must be carried by the seepage flow;
- There must be an unprotected exit (open, unfiltered), from which the eroded material may escape;
- For a pipe to form, the material being piped, or the material directly above, must be able to form and support "roof" for the pipe.

7.4.1 Piping in the embankment

Piping in dam embankments initiates by one of three processes: backward erosion, concentrated leak and suffusion (Fell, et. al, 2005).

Backward erosion - piping refers to the process in which erosion initiates at the exit point of seepage and progressive backward erosion results in the formation of a continuous passage or pipe.

Concentrated leak - piping involves the formation of a crack or concentrated leak directly from the source of water to an exit point and erosion initiates along the walls of the concentrated leak

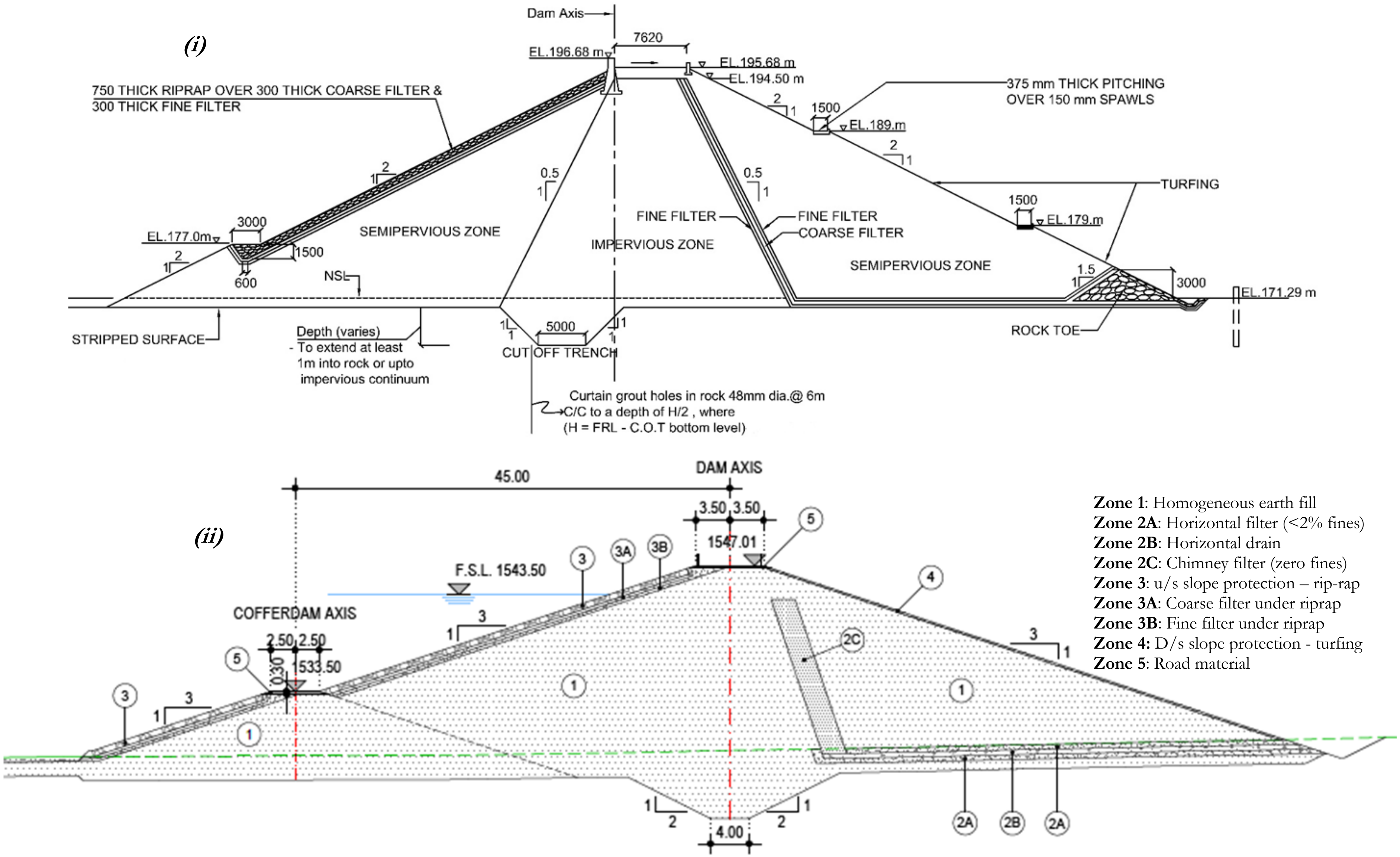

Figure 7-1: **(i)** Hirakud zoned earth fill dam (India); **(ii)** Warufu homogeneous earth fill dam (Rwanda).

Suffusion - involves the washing out of fines from internally unstable soils. Soils which are gap-graded, or which have only a small quantity of fine soil in a mainly coarse sand or gravel are susceptible to suffusion.

Figure 7-2 shows conceptual models or the development of failure for backward erosion piping and concentrated leak piping. The sequence of events leading to failure by the two models is essentially the same, however the mechanisms involved in the initiation and progression stages are different. Potential breach mechanisms include gross enlargement of the pipe hole; unraveling of the toe; crest settlement or sinkhole on the crest leading to overtopping; and instability of the downstream slope. Figure 7-3 shows a failure path diagram illustrating the possible sequence of events leading to dam breaching.

7.4.2 Piping through foundation

Piping in the foundation initiates by one of four processes: concentrated leak, backward erosion, suffusion, or blowout/heave followed by backward erosion. Figures 7-4 and 7-7 show the failure path diagram for piping through the foundation.

7.4.3 Piping from embankment into foundation

Piping from the embankment to the foundation involves backward erosion, or suffusion initiated by erosion of the embankment soil into open joints or open gravels in the foundation. Figure 7-8 shows conceptually the model for development of failure. Figure 7-9 gives a failure path diagram.

Table 7-1: Embankment dam typical construction materials (Fell et.al, 2005)

Zone	Description	Construction Material
1	Earth fill (impervious core)	Clay, sandy clay, clayey sand, silty sand, possibly with some gravel. Greater than 15% passing 0.075 mm, preferably more.
2A	Fine filter	Sand or gravelly sand, with less than 5% (preferably less than 2%) fines passing 0.075 mm & maximum size 75 mm.. Fines should be non-plastic.
2B	Coarse filter	Gravelly sand or sandy gravel, manufactured as for Zone 2A. Zones 2A and 2B are required to be dense, hard durable aggregates with similar requirements to that specified for concrete aggregates. They are designed to strict particle size grading limits to act as filters.
2C	Upstream filter and filter under riprap	Sand gravel/gravelly sand, well graded e.g. 100% passing 75 mm, not greater than 8% passing 0.075 mm, fines non plastic. Usually obtained as crusher run or gravel pit run with a minimum of washing, screening and regrading. Relaxed durability and filter design requirements compared to Zones 2A and 2B.
3A	Fine earth fill shell	Silty sandy gravel well graded, preferably with 2–12% passing 0.075 mm layer to reduce permeability. Obtained by crushing and screening rock or naturally occurring gravels or as crusher run.
3B	Coarse earth fill shell	Fine rockfill placed in 500 mm layers to result in a well graded layer sand/gravel/cobbles mix which satisfies filter grading requirements compared to Zone 3A.
4	Rock fill	Quarry run rock fill, possibly with oversize removed in quarry or on dam. Preferably dense, strong, free draining after compaction, but lesser properties are often accepted. Compacted in 0.5–1 m layers with maximum particle size equal to compacted layer thickness.
5	Rip rap	Selected dense durable rock fill sized to prevent erosion by wave action. In earth and rock fill dams often constructed by sorting larger rocks from adjacent 3A and 3B Zones. In earth fill dams either selected rock fill or a wider zone of quarry run rock fill may be used.

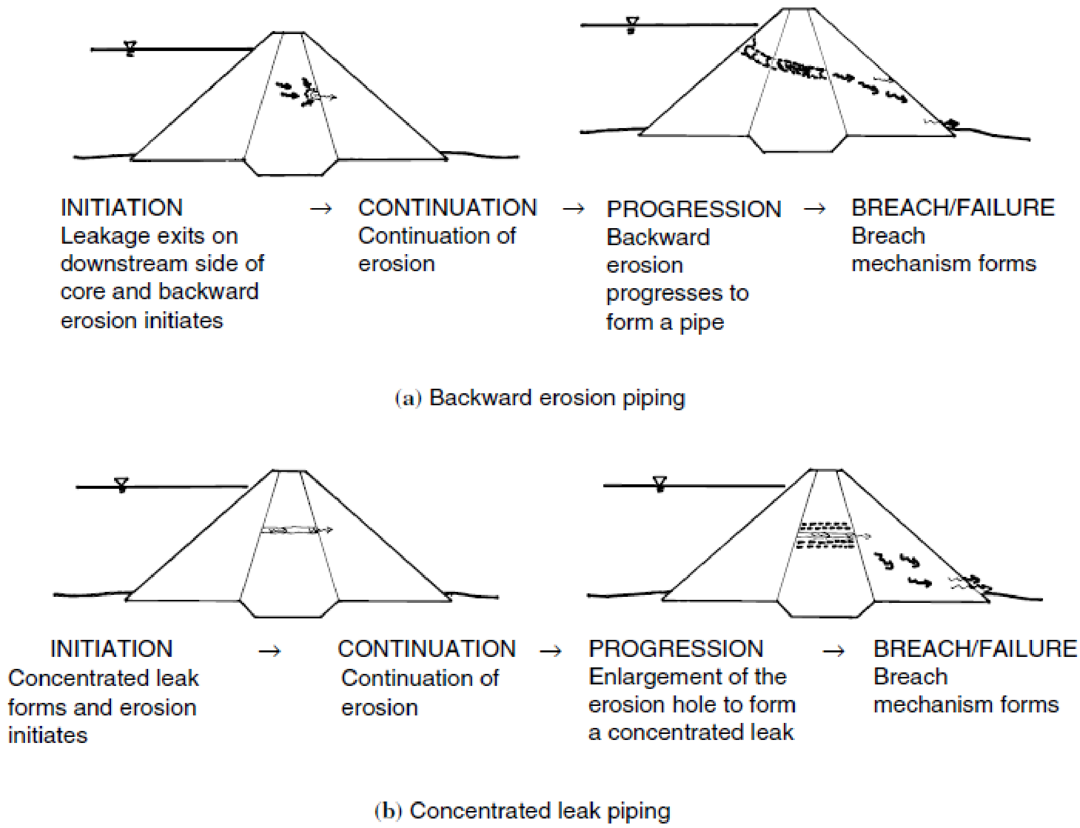

Figure 7-2: model for development of failure by piping in the embankment (Fell et.al., 2005).

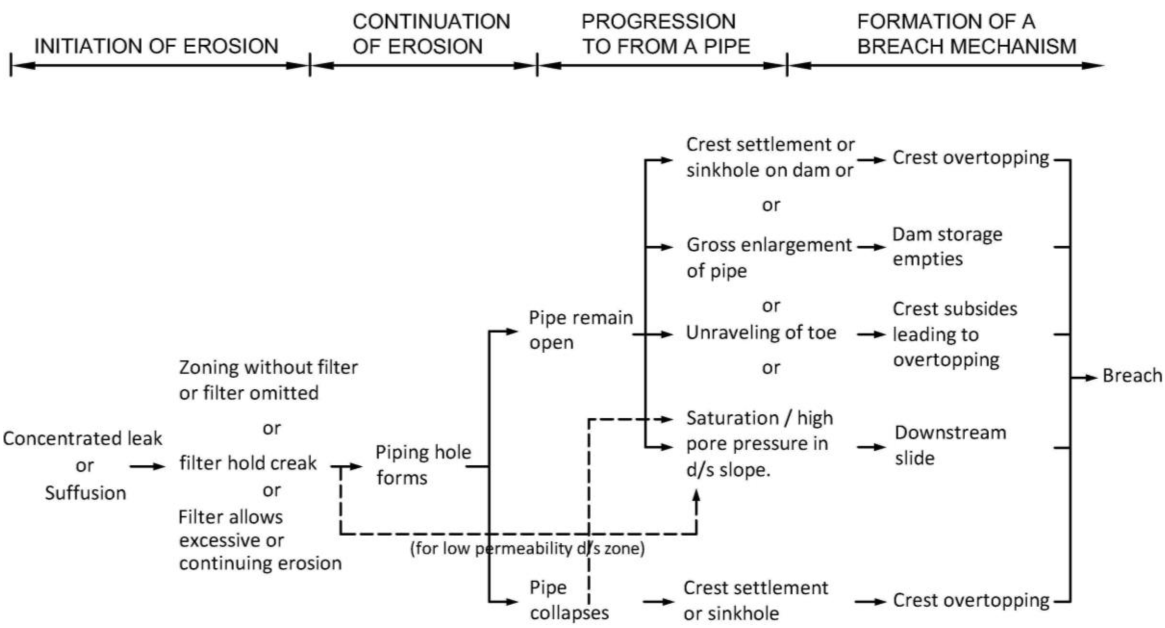

Figure 7-3: failure path diagram for failure by piping through the embankment (Fell et.al., 2005).

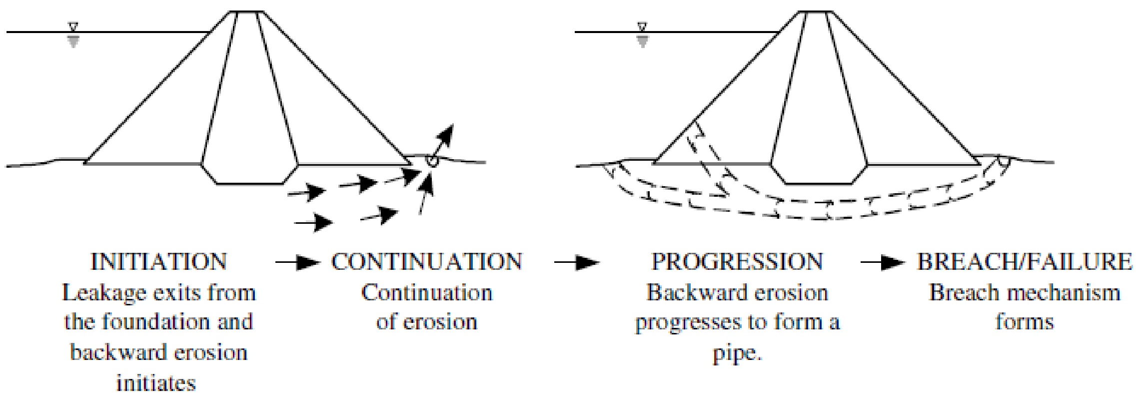

Figure 7-4**:** Model for development of failure by piping in the foundation (Fell et.al., 2005).

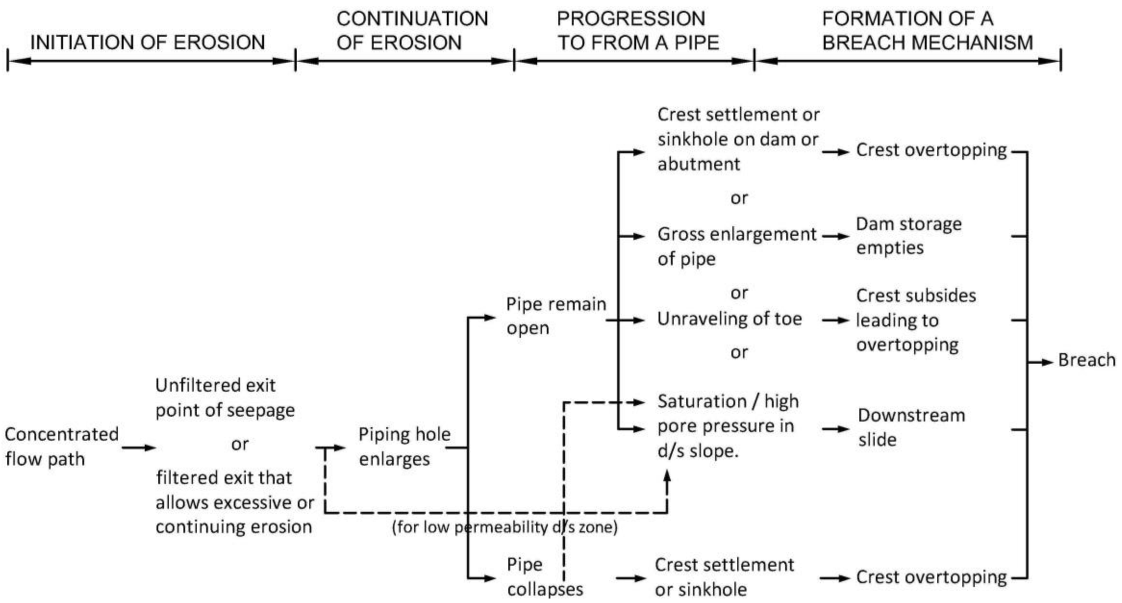

Figure 7-5: Failure path diagram for failure by piping through the foundation – concentrated leak and backward erosion piping (Fell, 2005).

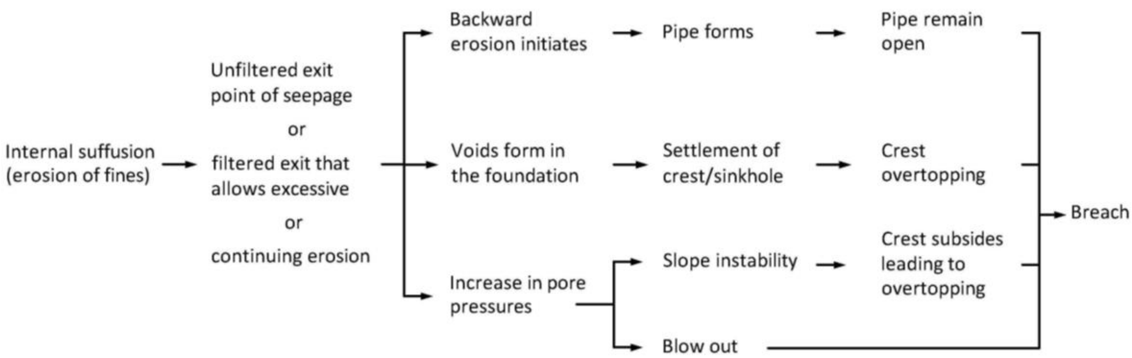

Figure 7-6**:** Failure path diagram for failure by piping through the foundation – suffusion (Fell et.al., 2005).

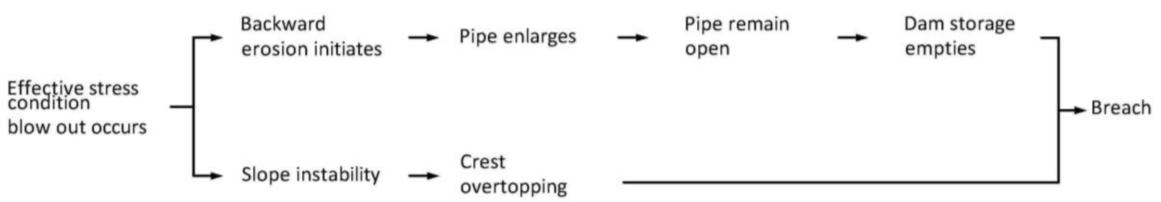

Figure 7-7**:** Failure path diagram for failure by piping through the foundation – blowout followed by backward erosion (Fell et.al., 2005).

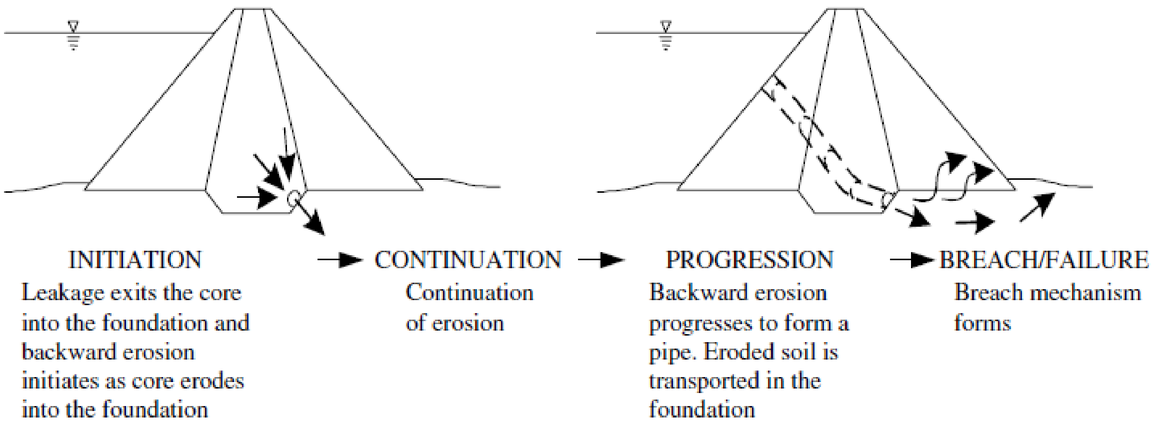

Figure 7-8**:** Model for development of failure by piping from the embankment into the foundation (Fell et.al., 2005).

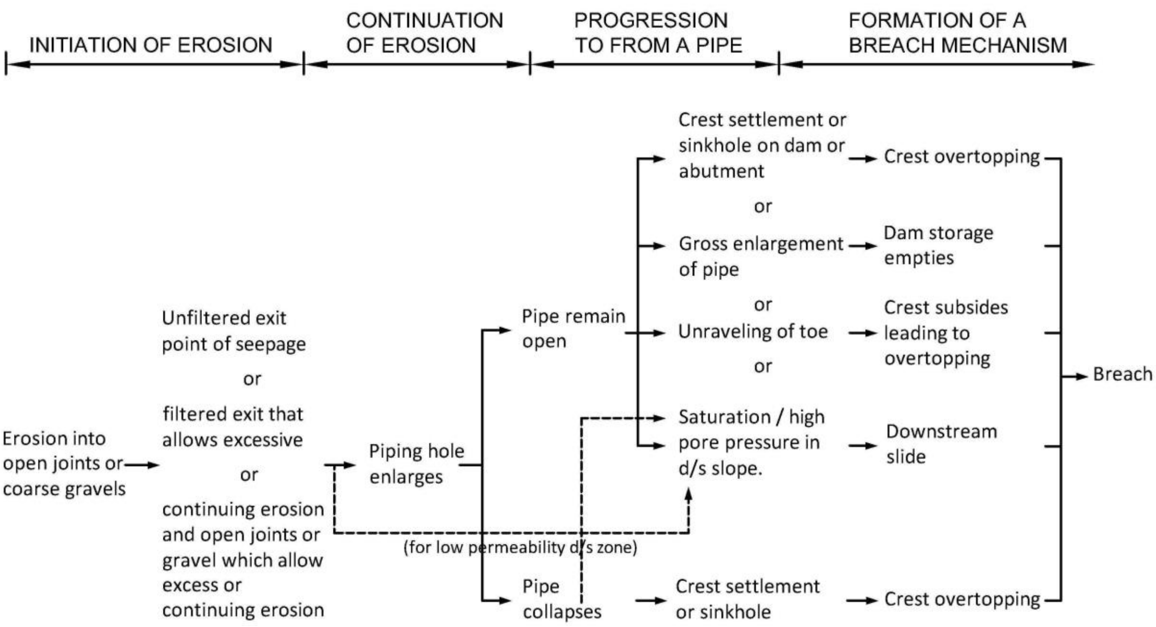

Figure 7-9**:** Failure path diagram for failure by piping from the embankment into the foundation – backward erosion piping (Fell et.al., 2005).

Table 7-2: Effect of design and construction details on the likelihood of internal erosion and piping in the embankment (Fell, 2005: Foster and Fell, 2000).

Factor	Relative importance			
	Initiation	Continuation	Progression	Breach
Geometry				
General zoning	L	-	M	H
Core width	M/H	-	L	L
Core width/height	L/M	-	M	L
Crest width	-	-	-	L/M
Freeboard	-	-	-	M/H
Downstream zone properties	-	-	M/H	H
Filter/code	-	H	H	H
Compatibility of dam core				
Classification	M	-	H	L
Erodibility/dispersivity	L	-	H	L
Compaction density ratio	M	-	M	L
Compaction water content	H	-	H	L
Permeability	M	-	M	L
Degree of saturation	M	-	H	L
Foundation				
Large scale irregularities	H	-	L	-
Small scale irregularities	M	-	L	-
Compressible soils	M	-	L	L
Conduits				
If present	H	-	H	L
Type joint/details	L	-	L	-
Settlement	L	-	L	-
Trench details	H	-	H	L
Walls abutting core				
If present	H	-	H	L
Slope	M	-	L	-
Collars/finish	L	-	L	-
Storage Volume	-	-	-	M
Closure section	M	-	M	L

Notes: (**1**) Relative importance weightings are judgmental and will vary from dam to dam.
(**2**) L = low, M = medium, H = high, – = not applicable.

7.4.4 Factors affecting internal erosion & piping

Table 7-2 gives a summary of factors that affect internal erosion and piping that can be used for assessing existing dams (Fell et. al., 2005; Foster, 1999; Foster and Fell, 1999b, 2000).

7.5 Stability Analysis

Stability analysis of embankment dams in India generally follow IS 7894. The analysis is usually carried out using Limit Equilibrium Methods. Limit equilibrium methods investigate the equilibrium of a soil mass tending to move down-slope under the influence of gravity. Forces, moments, or stresses tending to cause instability of the mass, and those that resist instability are compared. Two-dimensional (2-D) sections are analyzed and plane strain condition is assumed (i.e. strain in one direction is negligible or zero).

The shear strengths of the materials along the potential failure surface are governed by Mohr-Coulomb failure criterion.

$$\tau_f = c' + \sigma'_n \tan\emptyset' \qquad [7.1]$$

All limit equilibrium analyses use the method of slices, in which the soil mass above a trial failure circle is divided into a series of vertical slices as illustrated in Figure 7-10.

vertical slices of width b as shown in Fig. 7-11 (a). For each slice, its base is assumed to be a straight line defined by its angle of inclination θ with the horizontal while its

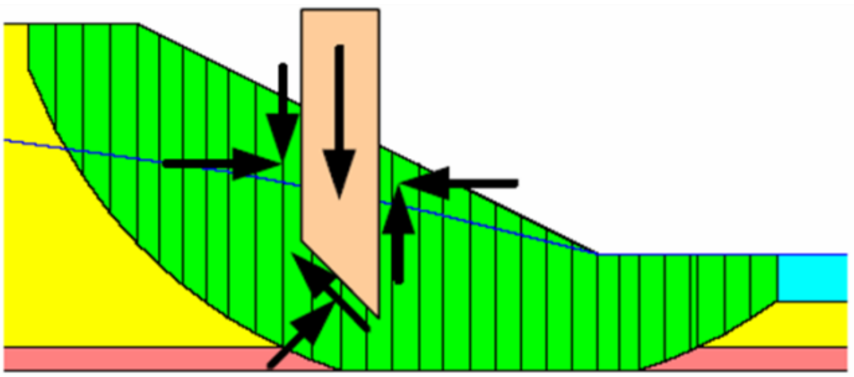

Figure 7-10: Slice forces in a sliding mass (note that the slice shown is in magnified form).

Requirements for static equilibrium of the soil mass are used to compute a factor of safety of the slope. The factor of safety is defined as the ratio of the available shear strength (τ_f) to that required for equilibrium (τ_m).

$$\boldsymbol{FS = \frac{\tau_f}{\tau_m}} \qquad [7.2]$$

At limit equilibrium FS = 1.0. A value of FS less than 1.0 indicates that the slope will be unstable with respect to sliding along the assumed particular slip surface.

In the method of slices, the soil mass above a trial failure circle is divided into a series of

height h is measured along the centerline of the slice.

Referring to 7-11 (b), the forces acting on a slice are:

W = total weight of the slice = $\gamma \times h \times b$.

N = total normal force at the base = N' + U, where N' is the effective total normal force and $U = u \times l$ is the force due to the pore water pressure at the midpoint of the base length l.

T = the mobilized shear force at the base = $\tau_m \times l$, where τ_m is the minimum shear stress required to maintain equilibrium and is equal

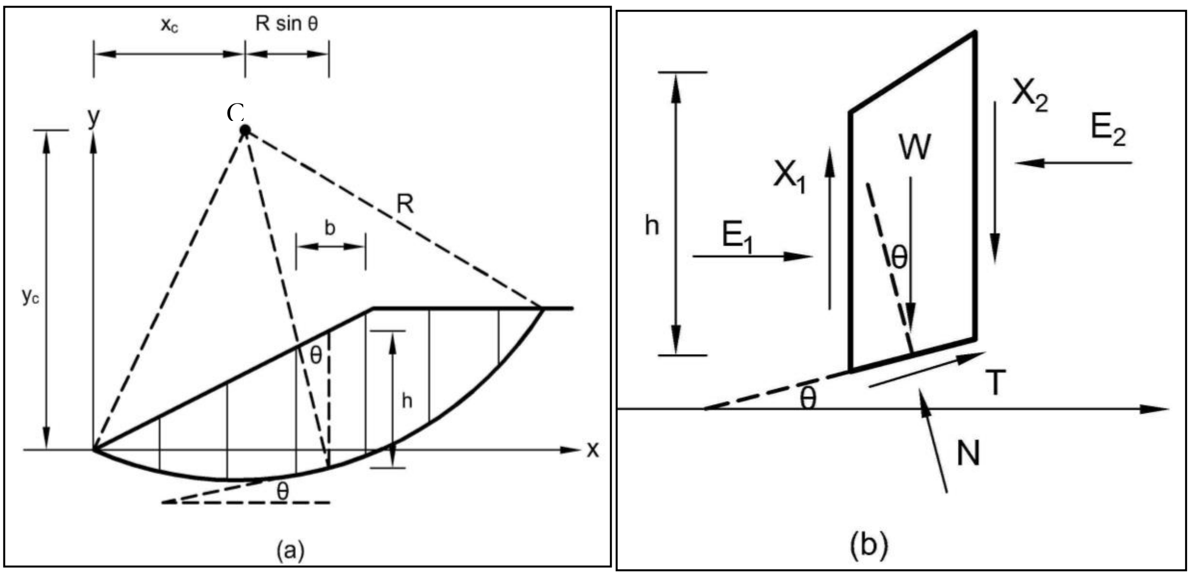

Figure 7-11: a) Method of slices; b) Forces acting on a slice.

to the shear strength divided by the factor of safety.

X_1, X_2 = shear forces on sides of the slice and E_1, E_2 = normal forces on sides of the slice. The sum of the moments of the interslice or side forces about the center C is zero.

Thus, for moment equilibrium about the center C (note the normal forces pass through the center):

$$\sum_{i=1}^{i=n} \boldsymbol{T_i R} = \sum_{i=1}^{i=n} (\boldsymbol{W}\sin\boldsymbol{\theta})_i \boldsymbol{R} \qquad [7.3]$$

Note also that inter-slice forces (shear & normal) at the interface of two slices are equal, collinear and opposite. Thus, summation of moments of these forces is zero.

As stated earlier $T_i = (\tau_m \times l)_i = (\tau_f \times l)_i / FS$, hence:

$$\boldsymbol{R}\sum_{i=1}^{i=n} \frac{(\boldsymbol{\tau_f l})_i}{\boldsymbol{FS}} = \boldsymbol{R}\sum_{i=1}^{i=n} (\boldsymbol{W}\sin\boldsymbol{\theta})_i \qquad [7.4]$$

Where n is the total number of slices. Replacing τ_f by the Mohr-Coulomb failure criteria we obtain:

$$\boldsymbol{FS} = \frac{\sum_{i=1}^{i=n}[(\boldsymbol{c'} + \boldsymbol{\sigma'_n}\tan\emptyset')\boldsymbol{l}]_i}{\sum_{i=1}^{i=n}(\boldsymbol{W}\sin\boldsymbol{\theta})_i} \qquad [7.5]$$

$$\boldsymbol{FS} = \frac{\sum_{i=1}^{i=n}[(\boldsymbol{c'l} + \boldsymbol{N'}\tan\emptyset')]_i}{\sum_{i=1}^{i=n}(\boldsymbol{W}\sin\boldsymbol{\theta})_i} \qquad [7.6]$$

The term $c'l$ may be replaced by $c'b/\cos\theta$. For uniform c', the algebraic summation of $c'l$ is replaced by $c'L$, where L is the length of the circular arc.

The values of N' must be determined from the force equilibrium equations. However, this problem is statically indeterminate – because we have six unknown variables for each slice but only three equilibrium equations. Therefore, some simplifying assumptions have to be made.

Two simple methods that apply different simplifying assumptions will be discussed in this section. These methods are called the Swedish method and Bishop simplified method.

The Swedish (also known as the ordinary) method of slices was introduced by Fellenius (1936). This method assumes that for each slice, the inter-slice forces $X_1 = X_2$ and $E_1 = E_2$. Based on this assumption and from statics, the forces normal to each slice are given by:

$$\boldsymbol{N} = \boldsymbol{W}\cos\boldsymbol{\theta} = \boldsymbol{N'} + \boldsymbol{ul}$$

$$\boldsymbol{N'} = \boldsymbol{W}\cos\boldsymbol{\theta} - \boldsymbol{ul} \qquad [7.8]$$

Substituting N' into Eqn. [7.2], we obtain:

$$\boldsymbol{FS} = \frac{\sum_{i=1}^{i=n}[(\boldsymbol{c'l} + (\boldsymbol{W}\cos\boldsymbol{\theta} - \boldsymbol{ul})\boldsymbol{tan}\emptyset')]_i}{\sum_{i=1}^{i=n}(\boldsymbol{W}\sin\boldsymbol{\theta})_i} \qquad [7.9]$$

For convenience, the force due to pore water is expressed as a function of W:

$$\boldsymbol{r_u} = \frac{\boldsymbol{u_i b_i}}{\boldsymbol{W_i}} \qquad [7.10]$$

Where r_u is called the pore water pressure ratio. Consequently, Eqn. [7.9] becomes:

$$\boldsymbol{FS} = \frac{\sum_{i=1}^{i=n}[(\boldsymbol{c'l} + \boldsymbol{W}(\cos\boldsymbol{\theta} - \boldsymbol{r_u}\sec\boldsymbol{\theta})\boldsymbol{tan}\emptyset')]_i}{\sum_{i=1}^{i=n}(\boldsymbol{W}\sin\boldsymbol{\theta})_i} \qquad [7.11]$$

The term r_u is dimensionless because the term $\boldsymbol{ub} = \boldsymbol{\gamma_w} \times \boldsymbol{h_w} \times \boldsymbol{b} \times \boldsymbol{1}$ represents the weight of water with a volume of $h_w \times b \times 1$. Furthermore, r_u can be simplified as follows:

$$\boldsymbol{r_u} = \frac{\boldsymbol{ub}}{\boldsymbol{W}} = \frac{\boldsymbol{\gamma_w h_w b}}{\boldsymbol{\gamma h b}} = \frac{\boldsymbol{\gamma_w h_w}}{\boldsymbol{\gamma h}} \qquad [7.12]$$

The height of water above the midpoint of the base is obtained by constructing the flow net. Alternatively, an average value of r_u may be assumed for the slope. By doing so it is assumed that the height of water above the base of each slice is a constant fraction of the height of each slice. If the height of the water and the average height of the slice are equal, the maximum value of r_u becomes $\boldsymbol{\gamma_w/\gamma}$, which for most soils, is approximately 0.5.

Note that the effective normal force N' acting on the base is equal to:

$$\boldsymbol{N' = W\cos\theta - ul = W(\cos\theta - r_u\sec\theta)}$$

If the term $(\cos\theta - r_u\sec\theta)$ is negative, N' is set to zero because effective stress cannot be less than zero (i.e. soil has no tensile strength).

The whole procedure explained above must be repeated for a number of trial circles until the ***minimum factor of safety corresponding to the critical circle is determined***. The accuracy of the predictions depends on the number of slices, position of the critical surface, and the magnitude of r_u. There are several techniques that are used to reduce the number of trial slip surfaces. One simple technique is to draw a grid and selectively use the nodal points as centers of rotation.

The Bishop simplified method (1955) assumes that for each slice $X_1=X_2$ but $E_1 \neq E_2$. These assumptions are considered to make this method more accurate than the Swedish method. An increase of 5% to 20% in the factor of safety over the Swedish method is usually obtained. Referring to Fig. 7.11 (b), and writing the force equilibrium in vertical direction (in order to eliminate E_1 and E_2), the following equation for N' can be found:

$$\boldsymbol{N'} = \frac{\boldsymbol{W - ul\cos\theta - \frac{c'l\sin\theta}{FS}}}{\boldsymbol{\cos\theta + \frac{\sin\theta\tan\emptyset'}{FS}}} \qquad [\mathbf{7.13}]$$

In addition to the force in the vertical direction, Bishop Simplified method also satisfies the overall moment equilibrium about the center of the circle as expressed in Eqn. [7.3]. Putting $l = b/\cos\theta$ and $ub = r_u W$, and substituting Eqn. [7.13] into Eqn. [7.6], we obtain:

$$\boldsymbol{FS} = \frac{\mathbf{1}}{\sum_{i=1}^{i=n}(\boldsymbol{W\sin\theta})_i}\sum_{i=1}^{i=n}\left[\frac{\boldsymbol{c'b + W(1 - r_u)\tan\emptyset'}}{\boldsymbol{m_\theta}}\right]_i \quad [\mathbf{7.14}]$$

where,

$$\boldsymbol{m_\theta} = \cos\boldsymbol{\theta} + \frac{\sin\boldsymbol{\theta}\tan\emptyset'}{\boldsymbol{FS}}$$

Equation [7.14] is non-linear in FS (that is *FS* appears on both sides of the equation) and is solved by iteration. An initial value of *FS* is assumed (slightly greater than *FS* obtained by the Swedish method) and substituted to Eqn. [7.14] to compute a new value for *FS*. This procedure is repeated until the difference between the assumed and computed values is negligible. Convergence is normally rapid and only a few iterations are required. The procedure is repeated for number of trial circles to locate the critical failure surface with the lowest factor of safety.

Hand calculation is possible for both Swedish and Bishop simplified methods. However, it will take weeks or months to calculate the minimum factor of safety for a large number of trial circles. It will even be more tedious if there are different materials in the dam and foundation with different soil parameters. Therefore, it is customary to use a computer program to carry out the stability analysis and determine the minimum factor of safety. For this purpose, software such as Geoslope, Slide, etc. are widely used by the Geotechnical engineering society all over the world.

In house software prepared by CWC & State Governments are also being used for the stability analysis of embankment dams.

Advanced methods recommended for practical use

Computers have made it possible to more readily handle the iterative procedures required in slope stability analysis. This lead to mathematically more rigorous solutions, which include all interslice forces and satisfy all equations of statics. Two such methods – commonly used in practice – are the Morgenstern-Price (1965) and Spencer (1967) methods. These methods are much more accurate but hand calculation using these methods is impossible.

There are several limit equilibrium methods available in the literature. The computer program SLOPE/W (Geoslope), for example, has included methods listed in Table 7-3 below. **As indicated in the table, the Morgenstern-Price and Spencer methods satisfy both force and moment equilibrium conditions and either of the two is recommended to be used in the stability analysis of** existing **embankment dams**. Experience shows that the value of the minimum factor of safety obtained from both methods is very close and higher than the ones obtained from Swedish and Bishop methods.

7.5.1 Load Conditions

As per IS 7894 and USBR, the loadings conditions to be examined should be based on knowledge of the reservoir operation plan, the emergency and maintenance operation plans, and the flood storage and release plan of the reservoir along with the behavior of the embankment and foundation materials with respect to the development of pore pressures in the dam and foundation.

The loading conditions to be examined for existing dams are:

a) Steady-state seepage condition: Steady state seepage conditions are usually assumed for the assessment of the long term stability of the downstream slope of the dam. The stability of the downstream slope should be analyzed at the reservoir level that will control the development of the steady-state phreatic surface in the embankment. This reservoir level is usually the full reservoir level (FRL) but may be lower or higher depending on anticipated reservoir operations. As per USBR, if the maximum reservoir surface is substantially higher than the FRL the stability of the downstream slope should be analyzed under maximum water level (MWL) loading. Pore pressures for the steady state seepage condition are estimated by calculating the flow-net for the embankment section either by graphical techniques or more commonly now by finite element methods (e.g. using SEEP/W software from GeoSlope).

b) Drawdown condition: When the reservoir level behind an embankment dam is lowered, the stabilizing influence of the water pressure on the upstream slope is lost. If the water level is dropped sufficiently quickly that the pore pressures in the slope do not have time to reach equilibrium with the new reservoir water level, the slope is less stable. It is often assumed, for design purposes, that the drawdown is rapid or even instantaneous. This assumption imposes severe loadings and it is often the controlling case for the design of the upstream slope. Accordingly, the upstream slope should be analyzed for rapid drawdown conditions from FRL to the minimum drawdown level (MDDL). The upstream slope should also be analyzed for rapid drawdown conditions from the FRL to an intermediate level if upstream berms are used.

c) Earthquake condition: The stability of existing dams should also be checked against earthquake loading. The effect of earthquake is considered by incorporating, in the stability analysis a static lateral force intended to represent inertia forces induced by the

Table 7-3: Limit equilibrium methods included in Slope/W (Geoslope) software.

Method	Moment Equilibrium	Force Equilibrium
Ordinary or Fellenius	Yes	No
Bishop's Simplified	Yes	No
Janbu's Simplified	No	Yes
Spencer	Yes	Yes
Morgenstern-Price	Yes	Yes
Corps of Engineers-1	No	Yes
Corps of Engineers-2	No	Yes
Lowe-Karafiath	No	Yes
Janbu Generalized	Yes (by slice)	Yes
Sarma-Vertical Slices	Yes	Yes

earthquake. This method of approach is termed as the pseudo-static analysis. In this method, effects of earthquake shaking are represented by accelerations that create inertial forces. These forces act in the horizontal and vertical directions at the centroid of each slice and are defined as:

$$F_h = \frac{a_h W}{g} = \alpha_h W \qquad [7.15]$$

$$F_v = \frac{a_v W}{g} = \alpha_v W \qquad [7.16]$$

Where a_h and a_v are horizontal and vertical pseudo-static accelerations; g is acceleration due to gravity; and W is slice weight. The ratio of a/g is a dimensionless coefficient α. In the slope stability analysis, the inertial effect is specified as α_h and α_v coefficients. These coefficients are considered as a percentage of g. For example, α_h coefficient of 0.24 means the horizontal pseudo static acceleration is $0.24g$. The horizontal inertial forces are applied as horizontal force on each slice as shown in Figure 7-12 below. For example if α_h = 0.24, the magnitude of the force is 0.24 times the slice weight.

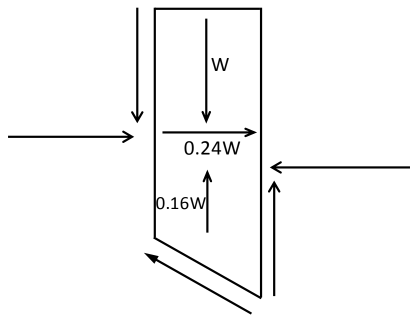

Figure 7-12: Horizontal and vertical inertial force at the centroid of a slice.

Vertical inertial forces are algebraically added to the slice weight. Depending on the earthquake shaking direction, vertical coefficient can be positive (downward against gravity) or negative (upward against gravity). The application of vertical seismic coefficients has often little impact on the safety factor, especially for frictional soil strength components. The reason for this is the vertical inertial force alters the weight of the slice. This alters the slice base normal, which in turn alters the base shear resistance. Hence, the added mobilized shear arising from the added weight tends to be offset by the increase in shear strength.

The vertical seismic co-efficient is normally taken as 2/3 of the horizontal seismic coefficient. The seismic zone is to be determined as per the Seismic map of India given in IS 1893 – 2002. The seismic coefficient to be used in pseudo-static analysis can be calculated as prescribed in IS 1893 – 1984. Where recommendations of National Committee on Seismic Design Parameters (NCSDP) are available, they may be adopted.

d) Construction condition: Generally it is not necessary to analyze during or end-of-construction stability for existing embankment dams.

7.5.2 Minimum factor of safety

Based on widely used international practices (e.g. IS 7894, USBR, USACE, etc.), for each loading condition a recommended minimum factor of safety is provided in table 7-4 below.

7.5.3 Shear strength parameters required for stability analysis

Steady state seepage loading condition should be analyzed using effective stress shear strength parameters in conjunction with estimated or measured embankment

Table 7-4: Loading conditions and factor of safety

Loading condition	Shear strength	Slope	Minimum factor of safety
Steady state seepage	Effective	Upstream, Downstream	1.5
Rapid drawdown	Effective or undrained	Upstream	1.3
Steady state seepage plus earthquake	Effective	Upstream, Downstream	1.1

and foundation pore water pressures. The use of Triaxial CD (Consolidated Drained) test to determine drained (c', $\emptyset'$) or Triaxial CU (Consolidated Undrained) test with pore water pressure measurement to determine both drained (c', $\emptyset'$) and undrained (c, $\emptyset$) shear strength parameters are appropriate. The CU test with pore water pressure measurement takes shorter time than CD test and is commonly used in practice. The use of the direct shear test is applicable for drained conditions with slow strain rate. However, direct shear test is not recommended to be used to determine undrained shear strength parameters as there is no way of sealing the specimen and drainage should be allowed throughout the test.

Types of shear strength tests required for stability analysis under diffecrent loading conditions recommended by IS 7894 are shown in Table 7-5 below. The same table also provides minimum factor of safety requirements for respective loading conditions, which generally agree with Table 7-4.

7.5.4 Pore-water pressure

Establishing the correct pore-water pressure is very important in the stability analysis of embankment dams. The most common way of defining pore-water pressure condition is using a piezometric line. Figures 7-13 and 7-14 show examples of piezometric lines (blue lines) for steady state seepage and rapid drawdown conditions, respectively of a zoned embankment dam composed of impervious core, pervious (or free-draining) shoulder, as well as vertical and horizontal filter/drain downstream of the impervious zone.

Note that if the shoulder zone in figure 7-14 is also of low permeability material (non

Table 7-5: Minimum Factor of Safety and Type of Shear Strength recommended in IS 7894

APPENDIX A

(*Clause* 5.1.1)

MINIMUM DESIRED VALUES OF FACTORS OF SAFETY AND TYPE OF SHEAR STRENGTH RECOMMENDED FOR VARIOUS LOADING CONDITIONS

Case No.	*Loading Condition of Dam*	*Slope Most Likely to be Critical*	*Pore Pressure Assumptions*	*Type of Shear Strength Test to be Adopted*	*Minimum Desired Factor of Safety*
I	Construction condition with or without partial pool*	Upstream and downstream	To be accounted for by Hilf's method	Q R†	1·0
II	Reservoir partial pool	Upstream	Weights of material in all zones above phreatic line to be taken as moist and those below as buoyant	R S‡	1·3
III	Sudden drawdown:				
	a) Maximum head water to minimum with tail water at maximum	Upstream	As given in 5.4.2	R S‡	1·3
	b) Maximum tail water to minimum with reservoir full	Downstream	As given in 5.4.5	R S‡	1·3
IV	Steady seepage with reservoir full	Downstream	As given in 5.5.2	R S‡	1·5
V	Steady seepage with sustained rainfall	Downstream	As given in 5.6.1	R S‡	1·3
VI	Earthquake condition:				
	a) Steady seepage	Downstream	As given in case IV	R S‡	1·0§
	b) Reservoir full	Upstream	As given in case II	R S‡	1·0§

NOTE — These factors of safety are applicable for the methods of analysis mentioned in this standard.

*Where the reservoir is likely to be filled immediately after completion of the dam, construction pore pressure would not have dissipated and these should be taken into consideration.

†This is to be adopted for failure plane passing through impervious foundation layer.

‡S test may be adopted only in cases where the material is cohesionless and free draining.

§Values are according to IS : 1893-1975 'Criteria for earthquake resistant design of structures (*third revision*)'.

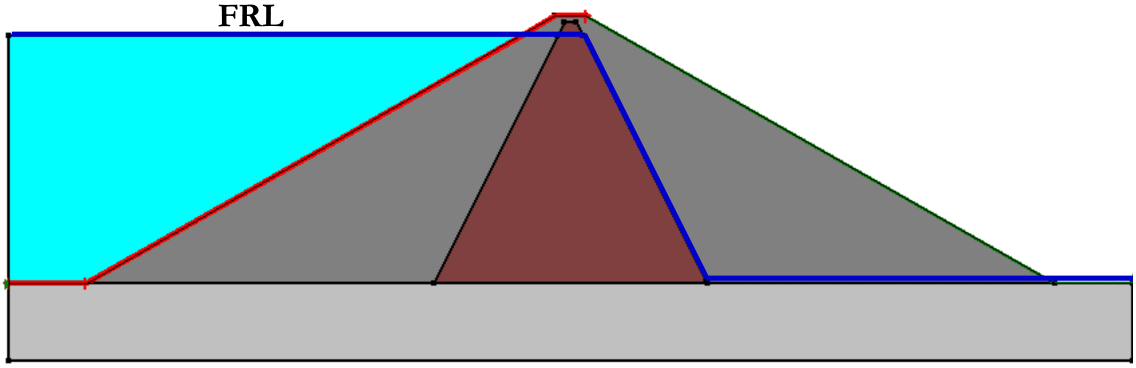

Figure 7-13: Piezometric line (blue color) for steady state seepage condition in a zoned dam.

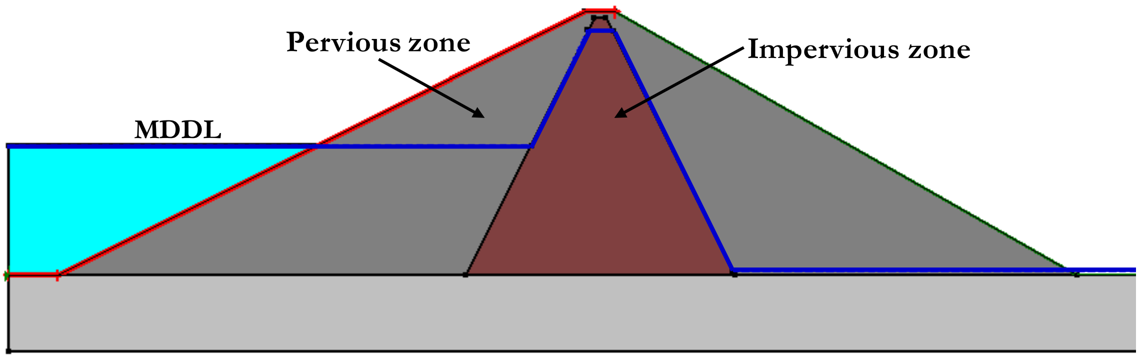

Figure 7-14: Piezometric line (blue color) for rapid drawdown condition in a zoned dam with free-draining shoulder zone.

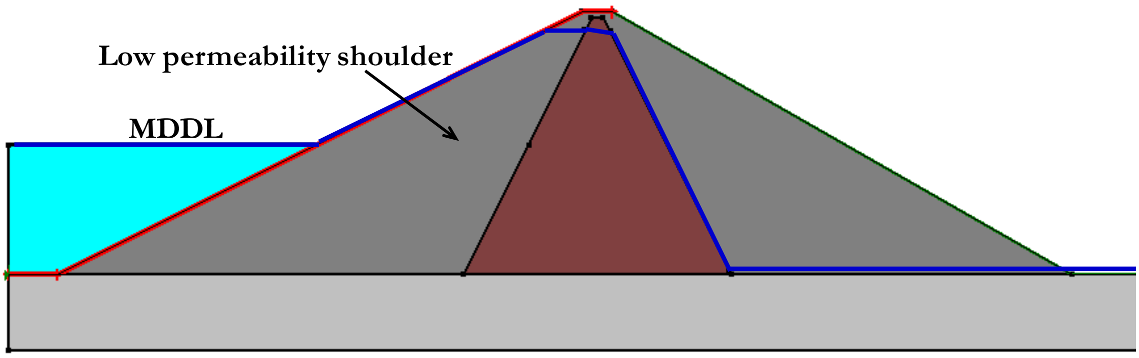

Figure 7-15: Piezometric line (blue color) for rapid drawdown condition in a zoned dam with low permeability shoulder zone (*all three figures above are prepared using Geolsope software*).

free-draining material), then during rapid drawdown, there will be pore-water pressure in this zone as well. Hence, the piezometric line will follow the upstream slope of the dam as shown in Figure 7-15.

To determine the piezometric line (also known as phreatic surface), Casagrande showed that the phreatic surface can be approximated by a parabola with corrections at the points of entry and exit. This simple method is given in Appendix I. However, current methods use Finite Element Method (FEM) based seepage analysis using state of the art software such as SEEP/W from Geo-Slope. In dams designed by CWC, it is also common practice to assume the phreatic surface line with a slope of 1V: 4H in the impervious core of a zoned dam up to the vertical chimney drain from where it drops down as shown in Figures 7-13 to 7-15. Also, to determine units weights of soils above and below the phreatic surface, the procedures described in IS 7894 can be used. An example of a stability analysis using Geoslope software is given in Appendix-J.

7.5.5 Considerations for dynamic analysis

As explained under section 7.5.1 above, seismic analysis of embankment dams generally use the pseudo-static method. This has been successfully used for reasonably well-built dams on stable soil or rock foundations and if estimated peak ground accelerations are not so high. However, in highly seismic areas and for dams involving embankment or foundation soils that may lose a significant fraction of their strengths under the effects of earthquake shaking, a dynamic analysis should be performed (USNRC, 1985). The main objectives of a dynamic analysis of embankment dams are assessment of **(i)** liquefaction potential of susceptible materials in the dam and foundation and **(ii)** determination of permanent deformations that will affect the freeboard of the dam.

Earthquake Data for Dynamic Analysis

Peak Ground Acceleration (PGA) **-** Following recommendations by the International Commission on Large Dams (ICOLD), two different earthquakes – the Maximum Design Earthquake (MCE) and the Operation Basis Earthquake (OBE) – shall be used for the dynamic analysis. The MDE is the largest reasonably conceivable earthquake that appears possible along a recognized fault or within a geographically defined tectonic feature, under the presently known or presumed tectonic framework (equivalent to 0.5% probability of exceedance in 50 years or approximately 1 in 10,000 years return period earthquake). The OBE is the earthquake which is expected to occur at least once during the expected life period of the dam (often accepted with 10% probability of exceedance in 50 years or approximately 1 in 500 years return period earthquake).

Under the OBE condition, structure of the dam should not be significantly impaired and should remain operational, even though some deformation is acceptable. For embankment dams, the MDE should not also cause the dam:

a) to lose its free board.

b) to fail due to liquefaction of material in the dam or its foundations.

c) to collapse due to movement at a slip

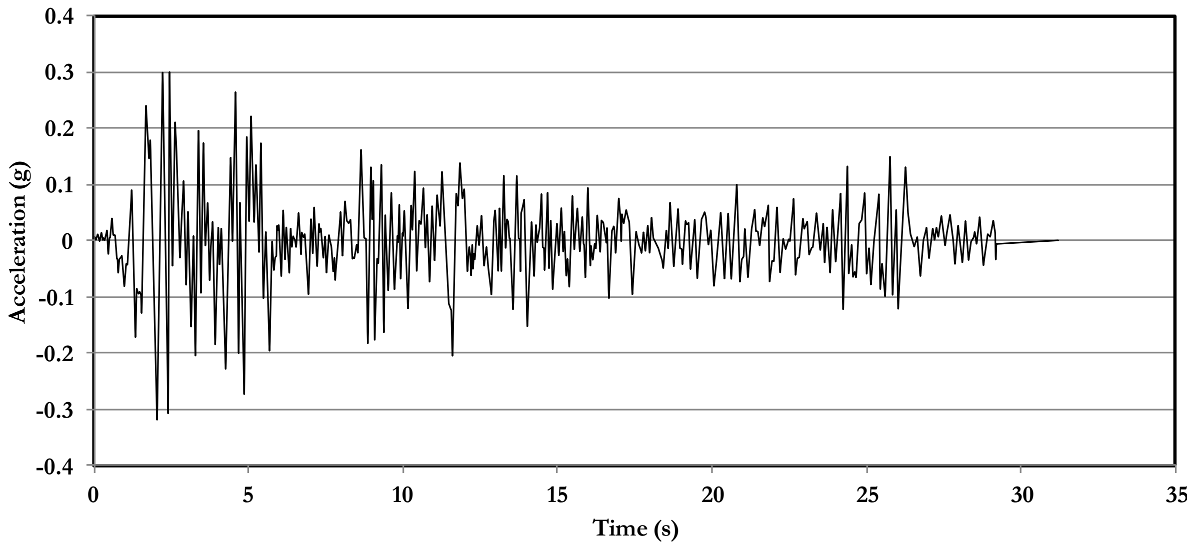

Figure 7-16: Recorded ATH for 1940 El Centro earthquake (www.vibrationdata.com/elcentro.dat).

surface in the slope or through the foundation.

Acceleration Time History (ATH) **-** Dynamic analysis of embankment dams are currently carried out using FEM based state of the art computer programs such as QUAKE/W from GeoSlope. Horizontal and vertical acceleration time histories are key input parameters for this analysis. Therefore, site specific horizontal and vertical ATH should be produced using the peak ground accelerations and available records of actual earthquakes.

An example of actually recorded ATH data for the 1940 El Centro earthquake is shown in Figure 7-16.

Liquefaction analysis - Liquefaction is one of the major effects of earthquakes, in which water saturated cohesionless soils temporarily lose strength and fail during shaking. The mechanism for this is, during strong shaking with no or limited drainage, cyclic shear stresses produce a progressive buildup of pore water pressure that significantly reduce the effective stress, which controls the strength of the soil. This pore water pressure development primarily depends on particle shape, size, and gradation. Most liquefaction is observed in clean sands. Well-graded soils are generally less susceptible to liquefaction than poorly graded soils. The first step to evaluate the potential of liquefaction is, therefore, identification of grain size distribution of the soil. Figure 7-18 shows grain size distribution boundaries separating liquefiable and non-liquefiable soils proposed by Tsuchida (1970) and widely used by geotechnical engineers worldwide. This figure can be used for the assessment of liquefaction susceptibility of dam and foundation materials.

Dynamic material properties - Dynamic characteristics of the dam and foundation materials such as shear modulus reduction and damping ratio functions need to be investigated by means of dynamic triaxial tests. As the dynamic shear strain increases, the effective dynamic shear modulus becomes smaller than the maximum value G_{max}. At the same time, the nonlinear response at higher dynamic strains leads to a higher rate of energy dissipation, which is represented by a

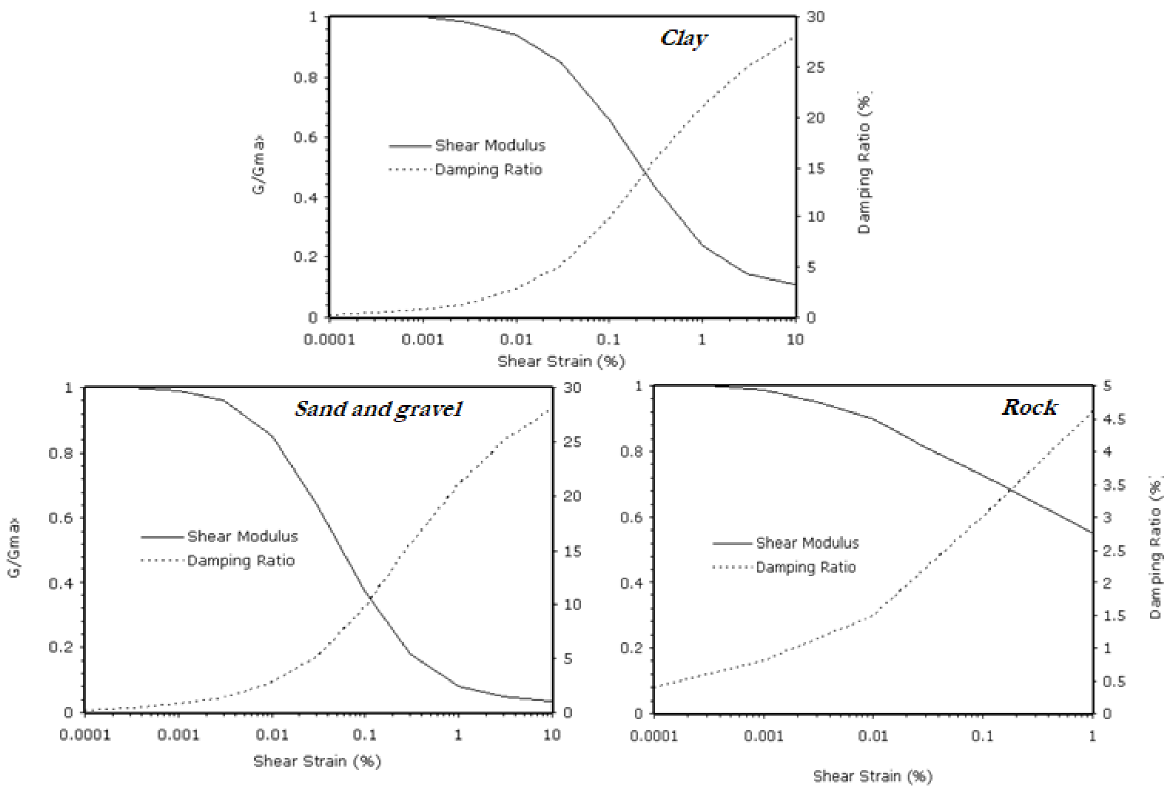

Figure 7-17: G-Reduction and Damping Ratio Functions (Sun et.al., 1988 and Idriss, 1980).

damping ratio that increases at higher strain levels. A lot of research has been done on this subject. For example, the strain-dependent dynamic shear modulus and damping ratio values have been published for different soils by Sun et al. (1988) and Idriss (1980) and summarized in Figures 7-17.

Other soil properties required for dynamic analysis of embankment dams include stiffness as a function of depth and cyclic pore-water pressure parameters. These and further details on dynamic material properties can be found in such reference as dynamic modeling with QUAKE/W (John Krahn, 2004, Geoslope).

Permanent deformation analysis - Examining potential permanent deformations resulting from the dynamic inertial forces is another important aspect of dynamic analysis of dams. From the view point of the earthquake safety of a dam, sliding displacements in the crest region need important consideration as they would lead to a reduction of the available freeboard. Such methods as the Newmark Sliding Block Concept are used to perform earthquake induced permanent deformations in dams. The Newmark method is based on the assumption that a potential sliding mass behaves like a rigid body, which would move down a slope as soon as the total (static and dynamic) driving force would exceed the available resisting force. Vertical displacements due to dynamic loading need to be added in the free board of the dam.

7.6 Embankment dam details

Riprap

Repair of displaced/disturbed upstream slope protection, i.e riprap, is among the most commonly implemented rehabilitation measures for existing embankment dams in India. For a successful repair, the rock size and thickness of the riprap need to be checked. Indian standard IS 8237 (1985) and USBR standard No. 13 (Chapter 7) are generally used for the design of riprap. The USBR method provides more details on how to calculate the required size and weight of rock as well as riprap thickness. An illustrative example based on USBR method is given in Appendix H. However, both USBR and IS Code 8237 can be used. Below the riprap, coarse and fine filter layers need to be provided. Requirements and design of filters are explained under the subsection below.

Filters and Drains

Filters and drains are critical sections of embankment dams. Filters/drains are necessary to avoid internal erosion, which might progress to form a pipe and breach the dam as well as to lower pore-water pressures in the dam and foundation such that there is an adequate margin of safety against slope in-

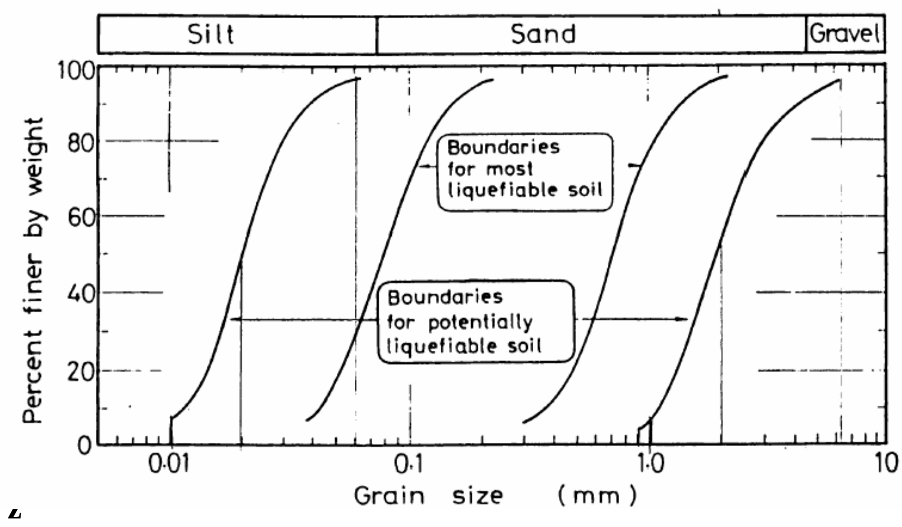

Figure 7-18: Boundaries separating liquefiable and non-liquefiable soils (Tushida, 1979).

stability. Some old existing dams may not have filter/drains and may require rehabilitation and provision of the same. In general, design of filters for embankment dams is based on IS Code 9429 (1999) and USBR (1987). The USBR method is based on Sherard and Dunnigan method (1985, 1989). The two methods are basically similar but more details are given in the later method. An illustrative example for design of fine and coarse filters is provided in Appendix G.

Filter Diaphragm

In some existing dams, excessive seepages have been observed at the dam toe, especially around conduits or outlet structures. To avoid piping including along and above the conduit, a filter diaphragm shall be provided to surround the conduit near its downstream end, i.e. underneath as well as on both sides and the top so that all potential leakage travels along the concrete-earth core interface exits in a controlled manner. Figure 7-19 shows a typical filter diaphragm for seepage and piping control around outlet pipe (Fell, et.al., 2005).

Downstream Slope Protection

Similar to upstream slope riprap, repair of eroded downstream slope is also among the most common rehabilitation measures implemented for existing embankment dams in India. As recommended in IS 8237 (1985) and based on other international practices, the downstream slope of earth-fill dams should be protected from erosion by providing turfing. Below the turfing, a 10 cm thick suitable soil (which is not part of structural requirement of the embankment dam) for grass growing shall be provided. This soil layer needs to be at least manually tamped and attain a reasonable degree of compaction to avoid erosion (in the range of 75% to 85%) and at the same time allow grass growing. In some cases anti-termite treatment may also be necessary in addition to turfing.

7.7 Repair of Cracks and Slip failures

Cracks - Wherever visible, cracks should be investigated in detail and appropriate remedial works be properly planned and carried out. Figure 7-20 shows longitudinal crack observed at Upper Mullamari dam in Karnataka, India. The cracks are recommended to be repaired by excavating a trapezoidal trench with 2V:1H side slope and minimum 1 m bottom width along the crack up to an appropriate depth (at least to the bottom base of the crack) and backfilled with compacted suitable soils. The backfill material should be compacted in layers with appropriate moisture content.

Slips failures - Some existing embankment dams have suffered slip failures on either upstream or downstream slope due to various reasons. Figure 7-21 below shows a slip failure on upstream slope of Willington earth fill dam in Tamil Nadu, India. The slip failure occurred on December 4, 2017 following heavy rains.

In such cases detailed investigation including soil testing and stability analysis should be carried out and rehabilitation measures implemented. The failed slope is recommended to be repaired by removing all the loose ma-

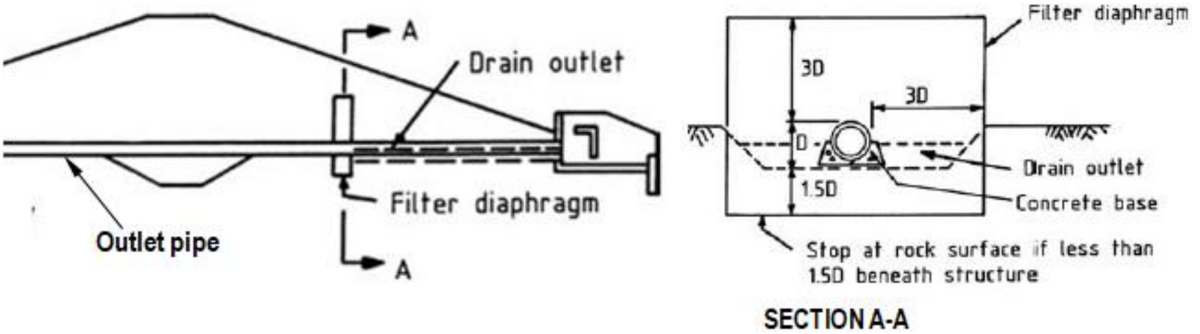

Figure 7-19: Filter diaphragm for piping control around outlet pipe (Fell et.al., 2005).

terial of the damaged portion of the dam; removing the earthwork in the shoulder section in benches to facilitate bonding and compaction in stages. Each bench should be excavated step by step. Next complete the earthwork by taking selected materials from the borrow pit area, backfilling the excavated portion, after proper rolling and watering. Then, bring the slope flatter than the original levels (usually 1V:3H) based on stability analysis). For upstream slope protection riprap & filters, and for downstream slope turfing to be duly provided.

7.8 Embankment Fuse Plugs

All existing dams need to be critically analyzed in terms of spillway design flood to ensure that the available spillway capacity is adequate to pass the design flood or that the expected short fall is made up with other suitable means, such as dam height raising, widening the existing spillway, or the provision of additional spillways. On existing dams, such measures may not be practical or may involve high investment costs, particularly because such additional capacity may be needed only for the floods of very low probability. It has been found that construction of a properly designed erodible fuse plug embankment is an economical alternative (*Hydraulics of Spillways and Energy Dissipators*, R. M. Khatsuria, 2005). For existing dams, only those locations where the emergency discharge would not endanger the main dam or other structures should be considered for fuse plug.

Figure 7-20: Longitudinal crack on d/s slope of Upper Mullamri dam, Karnataka.

Fuse plug, or a breaching section, is an erodible predetermined separate section of an earth dam designed to wash out when the inflow is in excess of the spillway capacity and the reservoir behind it reaches a specified level. The fuse plug collapses gradually over a reasonable time frame when overtopped, releasing surplus flood without endangering the safety of the main dam and lowering the reservoir level.

7.8.1 Selection Criteria

Topography - It is necessary to have a saddle at a reasonable distance from the main dam along the rim of the reservoir for discharging the excess flood through a natural or artificial tail channel into the same river or

Figure 7-21: Slip failure on u/s slope of Willington dam in Tamil Nadu.

a neighborhood valley.

Geology - A good quality rock should be available for the foundation of the fuse plug so as to withstand the erosive action of the flow when the fuse plug is washed out. If deep overburden exists in the saddle, it would be necessary to provide concrete cut-off walls beneath the fuse plug embankment to restrict the undermining of the foundation.

Downstream Condition - A suitable tail channel to lead the flow from the fuse plug into the main river should be available so that other adjacent structures are not endangered. The tail channel should be such that it would not be clogged by the eroded material from the fuse plug.

7.8.2 Design Considerations

There are only a few documented cases of fuse plug spillways actually operating. There is, therefore, general reluctance on the part of the designers to accept this type of device with confidence. The fuse plug also tends to get stabilized and compacted due to traffic, vegetal growth, and armoring over a long period. The principal features of a fuse plug spillway are as given below.

Pilot channel - It is provided in a short length of the fuse plug with it top level slightly lower than of the main breaching embankment fuse plug. It is designed to overtop and erode first. Thereafter the rest of the fuse plug is supposed to breach. Embankment material below the pilot channel should be of highly erodible nature to ensure effective washout of the fuse plug.

Impervious core - It is the key element in a fuse plug installation. It is a thin core inclined in the downstream direction. It prevents washout of the fuse plug for discharges smaller than the design flood and collapses when the pilot channel is overtopped.

Filters - The impervious core could dry and crack because the reservoir level may seldom reach that elevation. Suitable filters covering the core to prevent piping and premature washout of the fuse plug should be provided.

Sand and gravel - Sand and gravel form the major portion of the fuse plug embankment. The size and gradation of the material forming the fuse plug embankment influence the rate of washout.

Slope protection - Consisting of riprap and coarse gravel is provided both on the upstream and downstream of the fuse plug embankment to protect it against the action of wind, waves and rainfall.

A fuse plug should be designed as a zoned earth embankment dam and should washout in a predictable manner when overtopped. The washout of a fuse plug should begin at a pre-selected location and breach with reservoir water. When the reservoir level reaches a predetermined elevation, a low spot in the embankment called a pilot channel would be overtopped. By placing highly erodible materials in the pilot channel, breaching will occur rapidly and rest of the fuse plug embankment will washout without overtopping.

Appropriate zoning of the embankment is essential. The design section should have an impervious core inclined towards downstream, as shown in Figure 7-22. This is required so that when the material in the downstream zone is washed away, the overhanging impervious core breaks off under its own weight and the water load.

USBR (1985) carried out model studies on typical prototype fuse plug embankments varying in height from 3 to 9 m. Results of these model studies were published in REC-ERC-85-7 by USBR. From these studies, a series of gradation curves for embankment materials were derived and recommended for use in the prototype pilot channel and main section as shown in Figure 7-22. The gradation curves are also shown in the same figure.

Selected core material should normally comprise silt and/or clay. Filter zones should be provided both on the upstream and downstream of the core to prevent piping through cracks that might develop in the core. The compacted sand and gravel zone in the main fuse plug embankment and compacted rock–fill zone in the pilot channel section as shown in Figure 7-22, should be non-cohesive and easily erodible so as to initiate washout (R. M. Khatsuria, 2005).

7.8.3 Hydraulics of Fuse Plug

The flow through the breached opening of a fuse plug is similar to a flow over a broad crested weir. The flow over a horizontal broad crested weir is governed by:

$$\boldsymbol{Q = CLH^{3/2}} \qquad [\mathbf{7.17}]$$

Where Q is discharge, C is coefficient of discharge, L is length of the crest. Based on model studies conducted by USBR, the recommended values of coefficient of discharge C are: during washout in one direction 1.51, during washout in both directions 1.71, and after wash out is complete 1.44.

The lateral erosion rate (after the initial breach) for a given embankment design and flow depth has also been evolved from the model studies. Typically:

$$\text{ER} = \mathbf{14.6} \times \boldsymbol{H_f} + \mathbf{48} \qquad [\mathbf{7.18}]$$

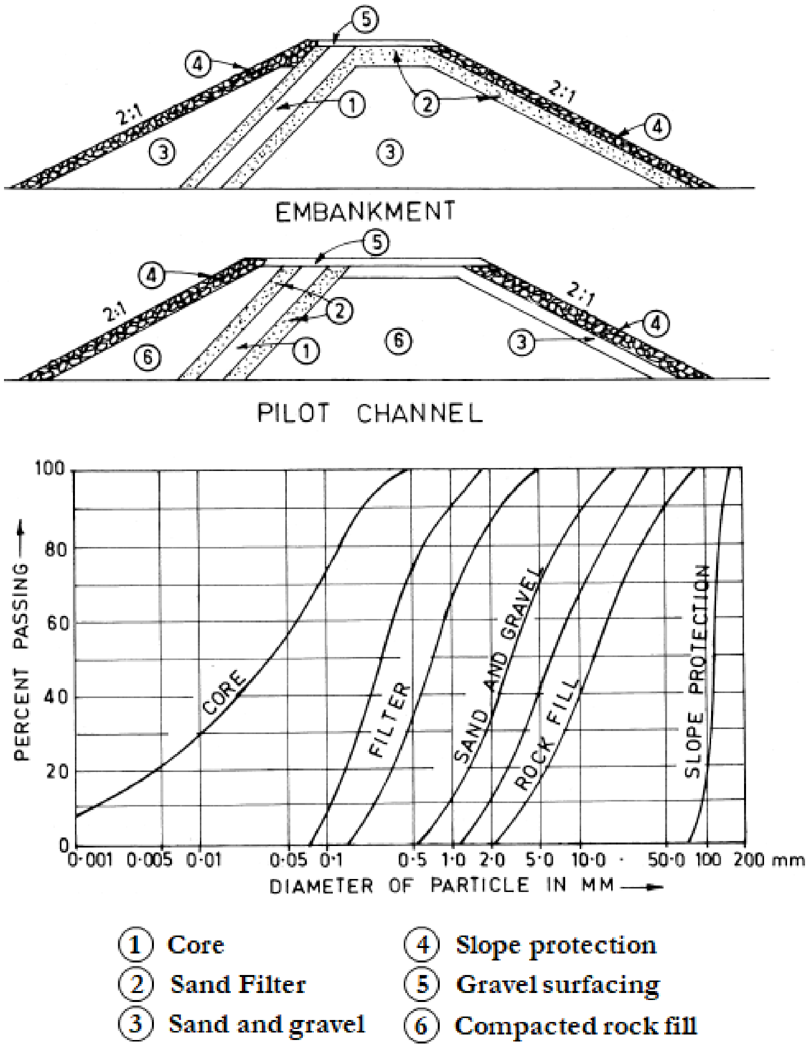

Figure 7-22: Zoning of materials and their gradation curves (USBR, 1985).

where

ER = Lateral erosion rate in m/hour.
H_f = Height of the fuse plug in m.

7.9 Embankment Dam Raising

In some existing embankment dams, it may be necessary to raise the dam height to accommodate a revised inflow design flood that exceeds the original design flood. In such case, the dam must be raised in a manner that will preserve the integrity of the structure. The dam raising design should consider the required increase in height, the minimum acceptable crest width, maximum embankment slopes, methods of achieving steeper than normal slopes, abutment contact areas, contact areas with appurtenant structures, and seepage control features (USACE, EM 1110-2-2300, 2004). The modified dam must be stable under all loading conditions including the design seismic event for the site.

7.9.1 Parapet Walls

The most cost-effective dam raising up to an increased height of 1 m is using a parapet wall (usually reinforced concrete). Although higher walls may be theoretically possible, this reflects the greatest height that will not interfere with visual observation of the upstream side of the dam from a vehicle on the crest (USACE, EM 1110-2-2300, 2004). Figure 7-23 shows a photo of conventional parapet wall (USACE, 2004). As per USBR Design Standard No. 13 (Chapter 13), for modifications of existing dams, par-apet walls should only be used to provide freeboard for wave run-up, not for wind setup or flood storage. This is because wave run-up is an intermittent type of loading, while setup and flood storage are constant loadings which could initiate seepage and erosion problems. Figure 7-24 and 7-25 show parapet walls constructed on dams in India.

7.9.2 Parapet Wall in Combination with Additional Embankment

If required dam raising is more than 1 m, USACE recommends that the cost-effective dam raising up to a height of approximately 4.5 m is accomplished using a 1 m high parapet wall in combination with a 2 to 3.5 m embankment crest raising. One such arrangement is shown in Figure 7-26 (USACE, EM 1110-2-2300, 2004).

Figure 7-23: Conventional parapet wall (USACE, 2004).

Figure 7-24: U/s and d/s parapet walls at Golwarpatti dam, Tamil Nadu, India.

Figure 7-25: Extension of u/s parapet wall at Kalo dam, Odisha, India.

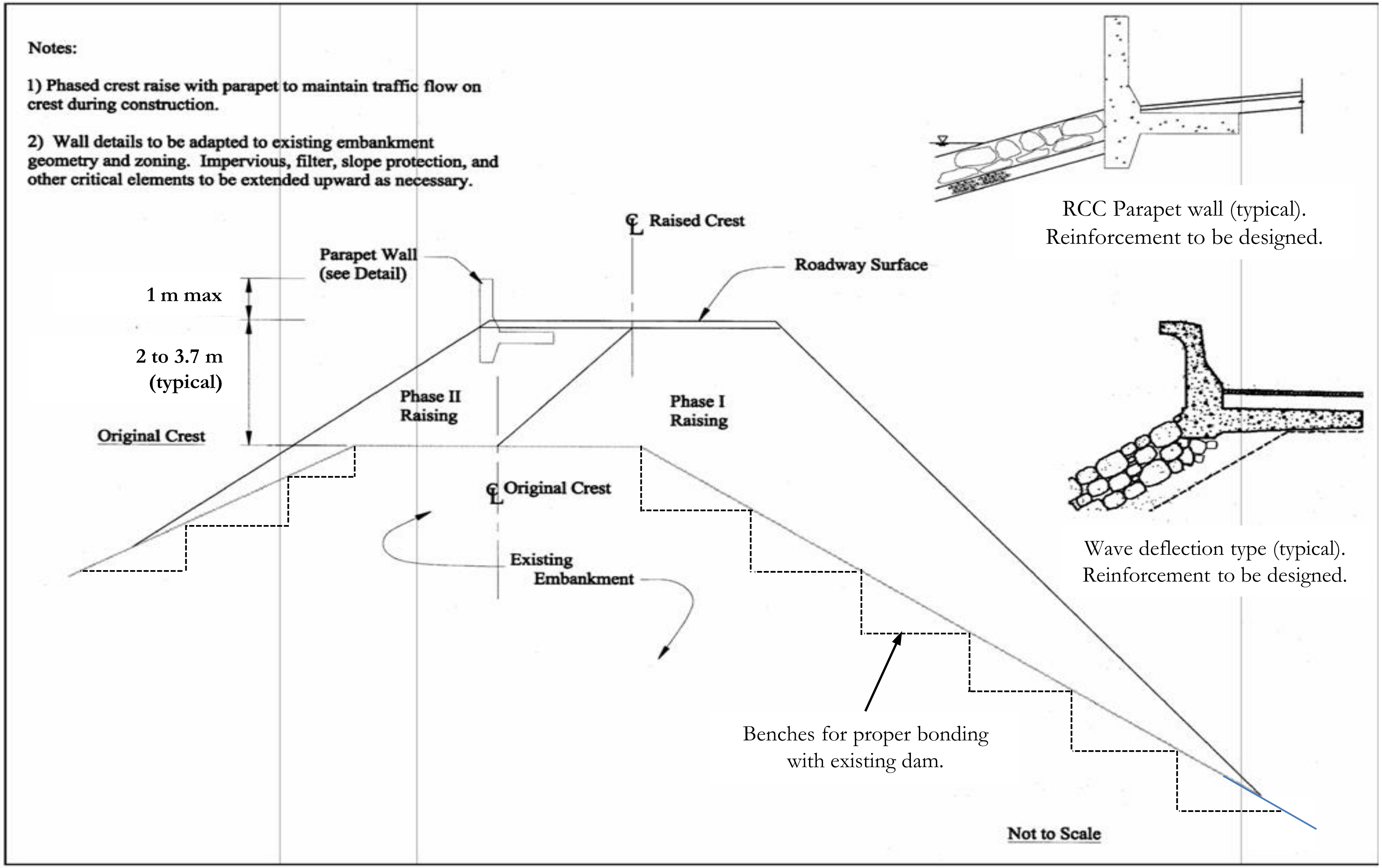

Figure 7-26: Embankment raising with parapet wall (USACE, EM 1110-2-2300, 2004).

Chapter 8. APPURTENANT WORKS

Various appurtenant works are required to be provided in a dam both for ensuring its safety as well as to cater to its operational requirements. Timely repairs and maintenance of these works (structures) is important to ensure safety of the dam and for drawing continued benefits from the dam.

The appurtenant works of the dam will generally include the following works:

1. Spillway and its allied works such as spillway piers, bridge, training/divide walls, energy dissipation arrangements (EDA) such as stilling basin , buckets (solid roller, slotted roller, ski- jump/trajectory buckets), downstream apron, plunge pool, downstream spill channel, any further energy dissipation works on its falls including retaining walls etc. as applicable to the dam.
2. Outlet works in an embankment dam or in the abutments or sluices in a concrete/masonry dam including trash racks, intake structure, gate shaft, approach bridge, outlet/sluice consisting of reinforced cement concrete conduit or a steel pipe either constructed or embedded in the dam, downstream energy dissipation structure etc.
3. Hydro-Mechanical Equipment's (gates, valves and hoists for the spillway, outlets, sluices etc.).
4. Galleries and adits in the dam, gate chambers, tunnels in the abutments/foundations, stair wells, elevator shafts containing passenger/ freight lifts etc.
5. Basic facilities such as approach / access roads, electrification in galleries, dam top and at all important points of the dam, Control rooms, Watch and ward rooms, Steps on the downstream face of Embankment dam etc.

8.1 Need for increase in Spillway Capacity

Over time, the estimates of inflow design floods have increased with improvements in technology/procedures and larger spillway capacities are required.

In the Indian context the inflow design flood to be considered is determined based on hydraulic head & gross storage at FRL in accordance with IS: 11223 Guidelines for fixation of spillway capacity.

Faced with the requirement to make dams safe in conformity with current practices the dam owners are examining both structural & non-structural options for the purpose.

The options that emerge from a study of the available modern case histories show that the following modifications are typical:

- Raising the height of a dam in view of higher maximum reservoir level.
- Constructing one or more additional (auxiliary) spillways, fuse plug/ breaching sections, flush bars etc.
- Provision of solid parapet wall on the upstream at dam top (where not available) provided that it is able to provide for the revised freeboard requirements.
- Strengthening the crest and downstream face of the embankment to allow some overtopping.
- Collecting more and better data to give advanced warning of adverse conditions and to monitor the response of the dam and reservoir.
- Lowering of the reservoir operating level to increase the flood storage volume.

- Modifying catchment flood characteristics by building flood detention devices or even an upstream dam.
- Increasing dam stability to accommodate higher flood water levels with cable anchors and mass gravity structures.

Further two or more dams located near one another in a series on the same river (a cascade of dams) are common at many locations in India. It is usual to consider the effect of the entire cascade in the revised design flood. From techno-economic consideration it may be desirable to increase storage and attenuation at one reservoir, thereby avoiding enlarging of spillways of downstream dams. However this exercise is required to be carried out at planning stage at the time of construction.

Care must be taken with gated structures so that adequate provisions are available to ensure that gates can be opened during a flood even when power may not be available. Operator trainings are absolutely necessary. Communication systems have often failed in an emergency emphasizing the need for effective training to ensure that operators can work effectively in isolation.

Under the DRIP it has been seen that the revised flood has increased in 183 dams out of 223 dams. Flood routing studies have been carried out to arrive at the revised MWL. Various structural & non-structural measures are being considered for the deficient dams.

The structural measures under consideration are:-

(i) Provision of additional spillways

(ii) Provision of flush bars & breaching dykes.

(iii) Increasing the dam height

(iv) Provision of up-stream solid parapet walls.

Before arriving at the final solution, various possible alternatives should be studied and the best one selected from techno-economic considerations.

Where ever additional spillways are proposed, first of all the suitability of the site will need to be established by necessary topographical and geological investigations which will be similar to those for new dams. The work will also envisage identification of quarries for construction materials, necessary material testing to establish their suitability for use, hydraulic design of the spillway and energy dissipation arrangements, verification of the hydraulic design by hydraulic model studies and their after detailed designs using various BIS codes and other technical literature.

The non-structural measures under consideration include:

(i) Lowering of the FRL

(ii) Flood Forecasting

(iii) Emergency Action Plan (EAP)

Some of the Bureau of Indian Standard (BIS) codes needed for the hydraulic and structural design of appurtenant works are given in the list of references at the end of this Manual.

8.2 Main causes of Damages in Spillways and other allied works

The main causes of damages in spillways and allied works are:

1. Erosion due to abrasion
2. Erosion due to cavitation
3. Erosion due to incorrect operation of gates
4. Scouring due to various reasons.
5. Obstruction by floating debris in the flow.

Most of these causes and remedial measures have also been covered in the Manual for Rehabilitating large dams. However a description of the same has been included in this Manual also (though repetitive)as these important issues are required to be studied and addressed while working out remedial measures, which may involve design studies, examination of the right type of materials for taking up remedial works and working out an appropriate methodology.

8.2.1 Erosion due to abrasion

8.2.1.1 General

Solid particles like suspended silt, rolling boulders, logs etc in the flowing water during monsoon can cause significant erosion by abrasion in hydraulic structures. The extent of damage depends on velocity of flow and intensity of discharge as well as the hardness of the abrasive material and the quality and nature of the surface being abraded. Spillways, stilling basins, bucket type energy dissipaters (especially slotted roller buckets), outlets etc. are particularly vulnerable to abrasion damages.

In India this problem is more in the Himalayan region where the silt load is quite heavy. In addition in many dams there is the problem of rolling boulders also. Damages are due to both abrasion as well as impact. Maneri dam & Ichari dam, Uttarakhand which are both being rehabilitated under DRIP are examples of these kind of damages.

Three principal types of abrasive action have been reported. The first is due to the suspended sediment load & sometimes rolling boulders which come along with flood waters & flow over the spillways, outlets etc. and over their energy dissipaters. The second type is the abrasion in energy dissipaters caused by rock/material drawn into them, especially in case of roller bucket (slotted roller or solid roller bucket), from downstream by reverse currents. The third is due the material that finds its way into stilling basins, tunnels or pipelines by other means, such as, construction or maintenance debris, fallen rock from side slopes etc.

Some stilling basins designed to form a hydraulic jump or Roller Buckets (Solid/ Slotted) tend to draw rock and sand from the downstream channel back into the energy dissipator and continue to circulate the material rather than eject it or sweep it from the basin. This circulation of sand and rocks is like the action of a ball mill, causing severe erosion of floors, side walls, floor blocks, and the bucket teeth. The depth of erosion may reach meters (ICOLD 1994).

Damages to low-level outlets and temporary diversion outlets have been reported in their conduit lining, gate and valve parts, and pipes. Particularly vulnerable are outlets used for diversion during construction, bottom outlets or outlets designed for the control of reservoir sedimentation.

Once damage to concrete or steel surface has started, abrasion accelerates with each operation of the spillway or bottom outlet. Hydraulic cavitation may also be triggered by the abrasion damage, increasing the rate of destruction.

Regular inspection of stilling basins after dewatering and low-level outlets is the only reliable means of detecting the extent of the damage. Underwater inspections have also been conducted. A case history is Potomac River No. 5 Dam (McClain et al. 1994).

8.2.1.2 Remedial Measures

Rehabilitation options for structures suffering from abrasion damage fall under three broad categories:

1) Repair of the damaged surfaces,

2) Re-design to prevent the flow conditions responsible for the damage, and

3) Use of improved operating techniques.

Abrasion damage can be repaired and minimized by constructing flow surfaces with special high strength concrete or resistant materials such as stainless steel.

Natural materials are often useful, particularly cut stone blocks of high-quality igneous rock (Kogovek 1997). Such cut stone blocks have been successfully used in Dakpather, Virbhadra and Asan Barrages in Uttarakhand.

However, these solutions are expensive and do not eliminate the cause of the damages. In the design of these facilities, it would be ideal to exclude the abrasive content of the flow as far as possible. But this is normally impracticable.

The power outlets are normally provided with sedimentation basins and are often required to be closed whenever the silt concentration in water during monsoon is high.

The use of high strength concrete and other resistant materials is normally recommended for the repair of heavily eroded areas. Silica fume in conventional concrete is an effective means of improving the resistance to erosion by surface abrasion. This extremely fine silica powder creates a hard and durable cementing paste in the concrete. Paste or mortar in concrete is susceptible to erosion by wear. Excellent quality hard aggregate will resist wear better than conventional aggregates. The combination of high-quality aggregate in silica-fume-modified concrete produces a harder and more durable material better suited to severe erosion environments. The High Performance Concrete (HPC) which is used for improving erosion resistance has been discussed in detail in the Manual for Rehabilitating Large Dams.

The performance of calcium aluminate cement and calcium aluminate aggregate have been reported by Cabiron (1996) and Cabiron and Lavignes (1998). Cylinder compressive strengths of as high as 50 MPa in 24 hours are reported, and the resulting material has shown in tests to be an effective repair material with excellent adhesion and durability under severe abrasion coupled with high water velocity.

Toyoda et al. (1991) report test results that show the resistance of a range of materials to attack by gravel. These showed that even high strength concrete can get eroded much faster than stainless steel and suggests that in the most severe cases more expensive solutions may be called for.

Under the DRIP, the repairs to spillway profile and slotted roller buckets of Maneri dam & Ichari dam in Uttarakhand are being carried out with high strength concrete.

Also where possible the topography d/s of the Energy dissipation arrangements can be modified by removal of obstructions to flowand rock protrusions in order to improve d/s flow conditions and to prevent drawal of materials into the energy dissipater on account of return flows/roller action.

8.2.2 Erosion due to cavitation

8.2.2.1 General

Cavitation is one of the most frequent causes of damages in high-head spillways and outlet works. Cavitation occurs in flowing water when a reduction of pressure within the water leads to a change of phase from liquid to vapor. The process starts with development of gas nuclei. The gas nuclei can grow rapidly, forming visible cavities in the fluid. As the local velocity increases, the pressure decreases proportionally, and it may reach a critical value at which the vapor cavities become unstable. The cavities collapse when they move into an area where the pressure is greater. The collapse of the cavities generates intense pressure shock waves, which produce noise and surface damage. The pressure bursts may reach thousands of MPa. The cavitation damage itself may produce a region of reduced pressure leading to further cavitation.

In gated outlets with large flow velocities, pressure reduction is primarily caused by changes in the local velocity caused by boundary irregularities including gate slots.

The most vulnerable area for outlets is the region where pressure flow changes to free-surface flow. This is normally downstream of a control gate or valve that discharges into a free-flow conduit, tunnel or a chute. Aeration is provided at such locations by suitably designed air vents.

Cavitation risk is usually based on the evaluation of the critical cavitation index, and its comparison with the cavitation number for the flow. Empirical expressions for the calculation of critical cavitation index are available in the literature. Such expressions were obtained by evaluating laboratory tests on flows across diverse types of surface irregularities including gate slots. The cavitation number for the flow in open channels or unpressurized tunnels depends on the vapor pressure, the steepness of the chute bottom, the radius of the vertical bend and water depth normal to the flow (Jansen, 1988). The cavitation number for the flow through gates depends on vapor pressure, water pressure and flow velocity at the critical location.

The potential for cavitation damage can be evaluated for existing hydraulic structures by measuring the pressure profile and comparing the local pressure with the water vapor pressure. Such field tests are applied during the phase of the design of corrective action for damaged structures. For practical purposes, the critical pressure for the onset of cavitation may be taken as the vapor pressure of the fluid. If the pressure within the flow fluctuates, there may be increased risk of cavitation, even though the mean pressure is well above vapor pressure.

As a rule, if the velocity of the stream entering a stilling basin exceeds about 20 m/s, the flow near baffle blocks may cavitate. Cavitation will be a serious possibility when the velocity exceeds about 25 m/s.

IS:4997 – Criteria for design of hydraulic jump type stilling basins with horizontal and sloping apron specifies that if the flow velocity is more than 15 m/s then the basin blocks should not be provided in a stilling basin.

As per ICOLD Bulletin 119 on Rehabilitation of dams and appurtenant structures it is now a practice to include aeration devices in spillways where velocities may exceed 20 m/s. The cavitation in the stilling basin of the Pit 6 Dam has been discussed by Cassidy (1994).

Tunnel spillways have been shown to be particularly susceptible to cavitation and care needs to be taken with both changes of grade and concrete finish to avoid creating zones of low pressure. In the tunnel spillway of Glen Canyon dam, USA an aeration slot was provided later to take care of damages due to cavitation. USCOLD 1996 may be referred to in this connection.

The effect of cavitation on concrete and steel surfaces can be unexpected, rapid and disastrous. In a single flood event, spillways have been destroyed, and valves or outlet works made inoperative. An example of the destructive force and speed of cavitation is found in the outlet at Tarbela Dam in Pakistan (Lowe et al. 1979). The flow velocity at which cavitation damage became significant was about 47 m/s in the tunnels.

8.2.2.2 Remedial Measures

Aeration of high-velocity flow has been proven to be the best method for the prevention of cavitation. Thus, the design of new structures should include aeration provisions to prevent or minimize cavitation damage.

IS: 12804 - Criteria for estimation of aeration demand for spillway & outlet structures can be referred to in this connection besides other specialist literature.

Existing structures that have been damaged by cavitation erosion may be retrofitted with aeration devices. In retrofitting structures with aeration slots, concrete may have to be dismantled/excavated. However, concrete excavation can be reduced using ramps to create a space for introduction of air.

The use of smooth walls and cavitation-resistant covering like high strength concrete, fiber concrete or steel can help in avoiding cavitation problems. Zhang (1994) has described work of this nature at Sanmexia Dam in China. However it is better to remove the cause of cavitation than to try to prevent the damage that cavitation causes.

In areas subject to erosion by both cavitation and abrasion, the use of special concrete or other materials should be considered. Concrete made of calcium aluminate aggregate and cement has been shown to be effective in resisting the effects of cavitation on concrete.

Fiber reinforced concrete includes between 0.5% and 1.5% by weight of cement of steel or polypropylene fibers. This amounts to about two million fibers per cubic meter of concrete. These fibers increase the toughness and tensile strength of the concrete. Impact resistance and fatigue strength are also improved. However, fiber-reinforced concrete is not as resistant to erosion by large water velocities as special concrete or blocks of igneous rocks. This is attributed to the grinding action of the sediment particles entrained in the water, coupled to flexure of the fibers leading to local damage of the surrounding concrete. Under cavitation conditions, the evidence is conflicting as to whether fiber-reinforced concrete is effective in reducing damage.

Patching can be used to correct damage from cavitation or erosion, initial tolerance errors, concrete form bolt holes, and lift-joint imperfections. In recent years, superior materials and procedures have been developed.

The patching material must be prepared for the needs of a given situation, and it must be properly applied and cured. Mirza and Durand (1994) report on a series of tests of repair materials and methods.

There is no simple solution for repairs. In some circumstances, the solution is to use concrete of the same quality as the surrounding material, and held in place monolithically. For this the repair material should have the same texture and thermal expansion / contraction characteristics as the surrounding material. However, even similar concrete placed after the original concrete has gone through its drying shrinkage can pull away from the base material as it cures. The designer may have to vary the properties of the patch material to account for this.

It is not necessarily true that high compressive strength means a better material. Crushing by compression is seldom the mode of failure in an environment of cavitation erosion. Failure is more often related to the dimensional stability, tensile capacity, fatigue endurance, strain capacity, and continuity of the repair material with parent material.

Epoxy resin is an excellent repair material, and yet many repairs made with epoxy resin have later failed. Investigation of failures has shown that the epoxy did not fail, but that the repair system did. That is, the epoxy itself held up well and was bonded to the concrete, but a separation occurred just below the glue line. Differing shrinkage or thermal properties can contribute to this occurrence. It also can be caused by vapor or water pressure building up beneath the epoxy, causing it to spall off along with the weaker concrete matrix just beneath it. In other cases, the epoxy repair has resisted the cavitation forces but transmitted them to the base material without redistributing them sufficiently for the core concrete to withstand them.

Monomers and polymers offer possibilities for repair work in concrete (Lampa, 1994). This type of material can also be used in original construction, to increase resistance to damage in areas where cavitation is known to be possible, or where expensive consequences are expected if damage did occur.

In new construction, the monomer can be soaked into the hardened concrete after moisture is force-dried out of the capillaries. It then is polymerized or solidified in situ. The resulting strength of the concrete and its resistance to cavitation can be significantly increased. The repair of Libby and Dworshak Dams used epoxy, fiber reinforced concrete and polymerized concrete (Regan et al. 1979).

For details regarding new materials, the Manual for Rehabilitating Large Dams may be referred to.

8.2.3 Erosion due to incorrect operation of gates

Asymmetric flow over the spillways especially in cases where a slotted roller bucket is provided for energy dissipation can cause considerable damages to the bucket teeth and the bucket.

As per IS 7365 Hydraulic Design of Bucket type of Energy Dissipaters it is necessary to operate all the spillway gates equally (under partial operation condition to achieve satisfactory performance of the bucket. Unsymmetrical operation of gates or operation of only a few gates at a time may set up horizontal eddies in the channel downstream which may bring debris into the bucket. All loose debris inside and just beyond the bucket should be removed after construction and before the bucket is put to use. The Indian standard further stipulates that divide walls to separate the flows would be necessary if unequal spillway operations cannot be avoided.

After witnessing damages in teeth of slotted roller bucket in many projects it is now a general practice to not to provide slotted roller buckets in any new projects.

A proper gates operation schedule tested through hydraulic model studies is desirable for all dam/barrage projects.

8.2.4 Scouring due to various reasons

8.2.4.1 General

Scour occurs due to the inter-action of fast flowing or turbulent water with natural materials like rock and soil in rivers/channels. The effect of impact, turbulence, and friction is to generate hydrodynamic forces against the faces exposed to the flow. Often these forces are not well understood, not properly accounted for and underestimated. As a result, hydraulic structures are sometimes under-designed in this regard and suffer considerable damage as a result.

Damages to stilling basins at Libby dam & Dworshak dam in USA occurred due to pulsating hydrodynamic pressures. These days' hydrodynamic pressures in a stilling basin are considered for the design of anchors below energy dissipaters as per IS 11527 - Criteria for structural design of energy dissipaters for spillways

Erosion downstream of a hydraulic structure is a result of local scour. Spillway structures discharging high flows with large velocities can result in large scouring downstream. Suitable precautions are necessary by way of provision of a cut off/key of suitable depth at downstream of the energy dissipater to avoid propagation of scour below the main dam structure. Structures on sand or soil foundations like barrages or at fall locations in spill channels for spillways located in flanks etc. are vulnerable to the erosion of the foundation. Also unlined chute spillways are prone to erosion/scour.

The potential erodibility of rock can sometimes be determined by precedent. However, erodibility is dependent on the geotechnical properties of the rock mass. A close study of the geology is essential to ensure sliding stability of dam along weak features especially where the rock foundation adjacent to spillway structure is eroded and the weak features are day lighted.

The technique of calculating stream power and comparing this with the Kirsten index appears to provide a useful tool for assessing erodibility (Van Schalkwyk et al. 1994).

Scouring is a complex phenomenon, and there are at present no analytical and experimental methods for forecasting scouring phenomena definitively. Physical Hydraulic Model Studies can be used in estimation of scour depths. There are empirical formulae also to assess scour depths in literature which can also be used.

Periodic visual or sounding surveys and underwater inspections by divers are useful in figuring out the extent and development of the scoured area. These inspections may have to be scheduled during non-monsoon periodically when there are no flows over the spillways. In some structures changes in normal operations may also be required to permit the surveys.

In the most severe cases, erosion downstream can undermine a major structure, causing structural collapse

8.2.4.2 Remedial Measures

Rehabilitation measures against erosion include two broad approaches.

First is to remodel the works/topography downstream of energy dissipaters by way of removing obstruction to flows due to which return/ unfavorable flows take place & their by improving the flow conditions.

In view of the proximity to the main structures blasting to remove rock protrusions is not recommended. Methods like diamond rope cutting, chemical splitting of rock, chiseling/wedging and barring, cutting rock by saw machine which do not envisage blasting are used for rock removal.

Second is to provide suitable protection works or to fill the erosion cavities with material more resistant to the eroding process.

Further in case of flip buckets these days plunge pools are being pre-formed to control haphazard scour.

There is also a problem just downstream of flip buckets where the low discharges fall and erode the foundation rock close to the bucket. As such protection works viz. RCC apron are generally necessary adjacent to such buckets. (Refer IS: 7365 - Criteria for Hydraulic Design of Bucket type Energy Dissipaters).

8.2.5 Obstruction by floating debris in the flow.

8.2.5.1 General

A frequent problem with overflow spillways and low-level outlets is the obstruction of the discharge by debris. This scenario has the most severe consequences when the spillway or the low-level outlet become inoperable. Trash racks can be damaged and the operation of gates and valves impaired by the debris.

Floating blockage can be a problem for overflow spillways. Logs can get stuck in partially open gates or in the gate/stoplog slots and prevent operation. Floating debris can also damage gates by physical impact.

In India this problem is faced in dams in the Himalayan region.

Low-level outlets, if not operated regularly, may get blocked not only by timber but also by sediment. Case histories of such rehabilitation works are Holmstyes Dam in the United Kingdom by Dyke et al. (1998) and Alloz Dam in Spain by Uceda et al. (1996). Clogging is more likely in small outlets that are infrequently operated. There are many such examples in India. Extended periods of time without operation may allow the openings to become permanently blocked and many lead to the loss of the facility. Silt accumulation in gate slots is a common nuisance. Valves that block the flow passage, such as butterfly valves and cone valves, appear to suffer more from blockage than valves or gates that expose the whole cross section.

The blockage of a spillway is often detected visually. Low-level outlet blockage is detected by the failure of the outlet to work. Frequent operation of gates and valves is recommended.

Siltation within a reservoir can be monitored by the survey of "silt lines" or by the observation of siltation at selected points. This is typically done using a Global Positioning System (GPS) to locate the points and soundings to measure the depth.

It is important to watch catchment conditions to predict when debris load is likely to be a problem. Forest fires can add significantly to the problem.

In the most severe cases, the spillway capacity may be reduced below the design requirement so as to endanger the dam from relatively small floods. Siltation or submerged debris may make bottom outlets inoperable and prevent the lake from being lowered in an emergency

8.2.5.2 Remedial Measures

The corrective action used for blockage prevention includes two broad approaches:

1) Removal of the solid material,

2) Adding measures that prevent it from obstructing the opening e.g. log boom, exclusive spillway for passing the logs etc.

Routine maintenance is an essential activity.

8.3 Outlet works

The operation of a dam depends on its outlet or control works to achieve its purpose of supply of water for irrigation, water supply, power generation etc. Typically, each outlet consists of an intake structure through which the stored water enters the outlet or a tunnel or a pipeline through gates/ valves.

Most of the operating components of outlet works (trash screens, control gates/valves, control systems) have a significantly shorter life than other elements of a dam. This is usually recognized at the design stage and facilities are incorporated to simplify replacement or repair. In some structures, the replacement method envisaged by the designer may require the water level to be lowered, often considerably. The loss of water and the resulting loss in revenue has encouraged many dam owners to seek solutions that do not need the reservoir water level to be lowered during the repairs.

Bottom outlets or scour outlets, where the intake is submerged by a considerable depth of water, pose a problem because of a significant amount of water loss, the time needed to lower and refill the reservoir, and sometimes the impracticability of doing this with uncontrolled inflows. A problem is the economic cost of the loss of water when the reservoir is emptied. This has prompted creative rehabilitation solutions to outlet works in which the work is done with the reservoir full.

Care is required in planning and carrying out work by divers. Even a water flow velocity of 2 m/s can make conditions extremely hazardous.

Provision of emergency gates/stop logs is generally made to facilitate repairs of service gates.

8.3.1 Outlet Tunnels and Conduits

The largest outlet tunnel repair was that undertaken at Tarbela Dam (Pakistan) where 382,000 m^3 of low strength concrete had to be placed in Tunnel No. 2 alone before construction of a new lining could begin (Chao 1980).

If there is cracking in the RCC outlet conduits it can be attempted to grout the cracks by a suitable grout material approaching from the d/s side with the main service gate in lowered position when the reservoir levels are low.

Sometimes rehabilitation of outlet conduits can be carried out by installing a small diameter sleeve within the existing outlet and grouting the annulus and the cracks in the outlet structure. All technologies used in conventional water pipeline replacement can also be considered, where feasible.

The design of high-head outlet conduits needs care. For long conduits, the downstream head loss will ordinarily produce the required back pressure to prevent cavitation, but for short conduits, gate passages often must be enlarged or exit constriction provided to generate appropriate pressure conditions. When conduits are flowing with entrance gates partially open, aeration from properly designed air vent is necessary because the back pressure will not be applied when the conduits flow partly full.

8.3.2 Bottom Outlets

The cavitation on bottom outlets can be minimized by the application of some or all of the following design features:

1) Improving the shape of water passages - Examples are streamlining of conduit entrances, increasing the amount of offset and decreasing the rate of taper downstream of gate slots, or using larger bend radii.

2) Increasing the pressure by raising the hydraulic grade line in areas of disturbed flow, which may be carried out by flattening any downward curve, restricting the exit end of the conduit, or increasing the cross-sectional area in such localities as gate passages to decrease the velocity and increase the pressure.

3) Introducing air into low-pressure areas not only to raise the pressure but to introduce air bubbles into the flow that will inhibit the formation of cavitation pockets and cushion the effects of their collapse.

Proper design reduces the probability of major problems occurring later. Large clear openings are required. Radial gates or slide gates are preferred (Lefranc et al. 1994). Trash racks, if they are used, should have a clear area of 60% to 90% of the opening. If hollow jet valves are used, narrow trash rack spacing is necessary to prevent the valves themselves from blocking.

Clogging of low-level outlets can be averted by suitable operation procedures in which the outlet valves or gates are routinely exercised, and the accumulation of debris near the outlet is removed. It is sometimes possible to flush debris through the openings. Emptying the reservoir is usually not an economical option, and underwater work by divers is required. Detailed collaboration with specialized divers is needed especially when there is a danger of underwater mudslides, or when they are working at depth.

Minimizing debris can be achieved by complete removal of vegetation from the bed of the reservoir before it is filled and periodic use of dredgers to remove sediments near the intake and the outlet. It also includes the use of dredgers to remove sediments near the intake of the outlet. Combined with the management of the catchment to prevent debris from entering the reservoir, this can reduce the risk of blocking of the outlet.

However small size low level river sluices (outlets) have not been very successful in Indian conditions as the sediment load is large & as they are not operated regularly.

Very often they become in-operable and are required to be abandoned.

8.4 Hydro Mechanical Works

8.4.1 General

Gates and valves in hydro projects are generally used to control the flow of water from the reservoir / pond of dams or barrages for various purposes such as passing the floods through the spillways or under sluices, release of water for irrigation, hydropower, water supply, navigation, ecological purposes, depletion of reservoir for inspection during emergency etc.

Hoists are used for operation (opening / closing) of gates as and when required.

Therefore, structural safety and periodic maintenance of gates and hoists is very important in any dam / barrage.

8.4.2 Classification of gates and Hoists

Gates may be classified based on

- Location of opening with respect to water head – Crest gates and Submerged gates
- Head of water – Low head gate (operates under a head of less than 15 m), Medium head gate (operates under a head of 15 m and above, but less than 30 m) and High head gate (operates under a head of 30 m and above)
- Operational requirements – Service gates, Emergency gates, Maintenance gates and Construction gates (e.g. diversion tunnel gate, construction sluice gate)
- Material used – Steel gates, Wooden gate, Cast iron gates etc.
- Mode of operation – Regulating gates and Non-regulating gates
- Shape – Hinged gates, Translatory gates and Multi-leaf gates
- Discharge through gates – Free discharging gates and Submerged flow gates
- Location of gates – Crest gates, Sluice gates, Flap gates, Diversion tunnel gates, Desilting chamber gates, Silt flushing gates, Head race tunnel gates, / Adit gates, Surge shaft gates, Penstock gates, Intake gates, Draft tube gates, Tailrace gates etc.
- Location of seal – Upstream seal gates, Downstream seal gates and Both upstream & downstream seal gates
- Location of skin plate – Gates with upstream skin plate and Gates with downstream skin plate
- Closing characteristics – Self closing gates and Gates requiring positive thrust for closure
- Drive – Manually operated gates, Electrically operated gates, Semi-automatic gates and Automatic gates

Hoists may be classified based on

- Type of drive - Manually operated hoist, Electrically operated hoist, Float operated hoist and Hydraulic hoist
- Operating mechanism – Mechanical hoist (rope drum hoist, monorail hoist, gantry crane etc.), Screw hoist, Chain & sprocket type hoist and Hydraulic hoist
- Mounting – Portable hoist (chain pulley block, winches, mobile crane etc.), Stationary hoist (rope drum hoist, screw hoist, chain hoist, hydraulic hoist etc.) and Moving hoist (gantry crane, E.O.T crane, monorail hoist etc.)

8.4.3 Most commonly used Gates and Hoists:

a) Crest gates

- Fixed wheel vertical lift gates/Radial gates – operated by rope drum hoist or hydraulic hoist

- Automatic gates – operated by float / counter weight operated hoist

- Stop log gates – operated by gantry crane or monorail crane with automatic engaging / disengaging lifting beam.

b) River sluice gates

Service or Emergency gate of Fixed wheel type or Slide type or radial gates or jet flow gates – operated by rope drum hoist/hydraulic hoist or screw hoist. Screw hoist is generally limited to 15T capacity.

c) Construction sluice & Diversion tunnel gates

Fixed wheel vertical lift gates – operated by rope drum hoist/chain pulley blocks/winches/mobile cranes.

d) Water Conductor System

- **Intake gates -** Fixed wheel vertical lift gates operated by rope drum hoist or hydraulic hoist or gantry crane for emergency intake gates

- **Desilting Chamber gates** : Fixed wheel vertical lift gates operated by rope drum hoist or hydraulic hoist or gantry crane

- **Silt Flushing gates** : Fixed wheel vertical lift gates or Bonneted type vertical slide gate operated by hydraulic hoist

- **Penstock intake gates -** Fixed wheel vertical lift gates or Bonneted type vertical slide gate operated by hydraulic hoist

- **Surge shaft gates -** Fixed wheel vertical lift gates - operated by rope drum hoist

- **Draft tube gates and Tailrace Outfall gates -** Fixed wheel type or slide type gates with provision of filling-in-valve – operated by rope drum hoist or gantry crane with automatic lifting beam

e) Canal system

- **Head regulator or Cross regulator gates -** Fixed wheel vertical lift gates or radial gates – operated by screw hoist, rope drum hoist or hydraulic hoist

- **Automatic gates -** Hinge type gates-operated by float or counter weight or Godbole type gates.

8.4.4 Design considerations of commonly used Gates and Hoists

8.4.4.1 Fixed wheel gate

Fixed wheel gates are most commonly used in dams/barrages, canal structures etc. General design of fixed wheel gate involves design of the following components:

a) Skin plate

b) Vertical/horizontal stiffeners and main horizontal girders

c) Wheels and wheel tracks & track base

d) Seals & accessories and seal seat & seal base

e) Guide rollers/guide shoes and guides

f) Sill beam

g) Anchors and Embedded parts (1st stage anchors & 2nd stage embedded parts)

h) Gate lifting attachments

i) Dogging beam/Latching arrangement

Fixed wheel gate is designed as per IS 4622. Skin plate is normally designed as panel construction. Skin plate and stiffeners are designed together in composite manner. To take care of corrosion, the actual thickness

Figure 8-1: Fixed Wheel Vertical Lift Gate

of skin plate is provided at least 1.5 mm more than the theoretical thickness computed. The thickness of the skin plate shall not be less than 8 mm inclusive of corrosion allowance.

The horizontal/vertical stiffeners are designed as simply supported or as continuous beams depending upon the framing adopted for gate. The spacing between main horizontal girders shall preferably be such that all the girders carry almost equal loads.

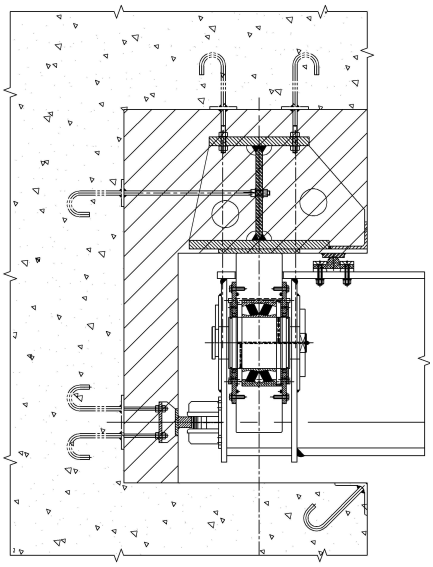

Figure 8-2: Groove Detail for Fixed Wheel Gate

The end vertical girders are designed as continuous beams resting on wheel centre points with concentrated loads, coming from horizontal girders, at points where they meet the end vertical girders.

Maximum deflection of the gate under normal conditions of loading shall be limited to L/ 800 of the span (c/c of wheels).

The wheels for the fixed wheel gates are usually made of cast steel or forged steel. The wheel pin transmits the load from the end verticals to the wheel through antifriction spherical roller bearings. The wheel / roller track is the load bearing member on which the wheels of fixed wheel gate transfer the thrust on account of water pressure. The wheel track transmits the heavy loads from the wheels on to the concrete piers / abutment without allowing the stress in concrete to exceed the permissible bearing & shear stress in pier.

The wheel track shall be provided in a true vertical plane and shall have the smooth machined surface for the wheels to travel / roll over and transmit the loads through the tracks to supporting concrete. The hardness of wheel track surface shall be kept minimum 50 points BHN higher than that of wheel tread to reduce wear & tear of track.

The wheels are designed with either point contact for curved treads or line contact for plane treads depending upon the contact surface geometry of the wheel.

The contact of all the wheels (with more than four wheels on the gate) with the track can be ensured if the gate is designed as semi-flexible connection among the end vertical plates of the multiple element construction gate with only two wheels on either side in each unit of the gate. The elements of gate are joined by splicing with rubber gaskets on either side of the skin plate and pin joint on end vertical girders. The wheel pins are normally provided with 5 mm eccentricity to permit proper alignment of wheels.

The gate shall be reasonably water tight. The maximum permissible leakage should not be more than 5 litres / min / metre length of seal in case of low and medium head gates and 10 litres / min / metre length of seal in case of high head gates. Rubber seals are provided on the gate to prevent leakage. For reducing seal friction, fluorocarbon / Teflon cladded seals may be used. The initial interference of 2 mm to 5 mm is provided for double stem or music note type seal depending upon the requirement and type of installation.

Guide rollers / shoes are provided on the sides of the gates to limit the lateral movement of the gate to be not more than 6 mm in either direction. A minimum load of 5 % of the total dead weight of the gate is recommended for the design of each guide roller.

The sill beam is provided with corrosion resistant steel flat welded or screwed with corrosion resistant steel screws and flush with crest face/profile. Gate bottom with bottom rubber / neoprene seal shall rest on the sill beam.

In case of medium and high head gates, the bottom shape of the gate shall be suitably designed to minimise down pull in the case

Figure 8-3: Caterpillar type Vertical Lift Gate

of downstream sealing and to minimise uplift and vibrations in case of gates with upstream sealing. Such gates should be designed to provide a converging fluid way and streamlined flow which may require suitable recess and provision of an air vent.

The plate with 'J' or 'U' anchors shall be provided in the first stage concrete, for fixing 2nd stage anchors for alignment of embedded parts in the groove area, with suitable block out openings, to hold the embedded parts till the second stage concreting is completed & cured. The minimum size of first & second stage anchor bolts shall not be less than 16 mm diameter and the anchor plate thickness shall not be less than 8 mm.

The gate components are designed for wet condition (accessible or inaccessible) or dry condition (accessible or inaccessible) depending upon their component part configuration. Allowable stresses for design of various components are considered based on the conditions whether dry or wet and accessible or inaccessible.

For gates without top seal, suitable free board over Full Reservoir Level (FRL) is to be provided while deciding the gate height.

Silt load shall be considered in design of gate wherever applicable as per provision of Indian standard.

The gate is checked for earthquake effect according to the seismic zone of the project. The gate is also checked for Maximum Water Level (MWL) condition. The allowable stresses are increased by 33.33 % of the values specified in IS: 4622.

The gate is normally designed to close under its own weight with or without addition of ballast, but under certain conditions gate may require a positive thrust for closing where hydraulic / screw type hoists providing positive thrust are used.

Distance between two gate slots in a pier / wall should be sufficient to provide individual hoists for gates, otherwise only a common hoist platform is to be provided.

Air vent pipe of suitable size should be provided in the concrete piers / structure as per requirement wherever applicable for adequate aeration to avoid cavitation.

Dogging beam or latching arrangement is used for dogging or latching of gate at deck level when not in use or during maintenance of gate.

8.4.4.2 Radial gate

Radial gates are most commonly used at dam spillways. General design of the radial gate involves design of the following components:

a) Skin plate and stiffeners
b) Horizontal girders
c) Arms and bracings
d) Trunnion hub, pin and bush or bearing
e) Trunnion brackets
f) Trunnion girder or yoke girder
g) Load carrying anchors
h) Anchorage girder
i) Thrust block or Trunnion tie (if inclined arms are used)
j) Seals, Seal seat, seal base and Sill beam
k) Wall plate and Guide rollers
l) Anchor bolts
m) Gate lifting attachments
n) Dogging beam / Latching arrangement

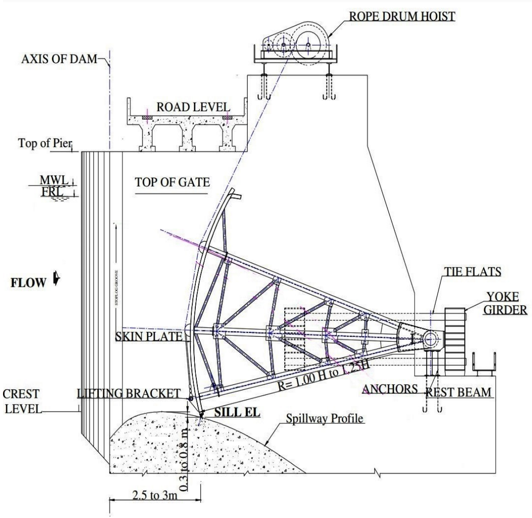

Figure 8-4: Spillway Radial Gate with Rope Drum Hoist

Radial gate is designed as per IS: 4623. Skin plate and stiffeners are designed together in composite manner. The stiffeners may, if necessary, be of a built-up section or of standard rolled steel section. The thickness of the skin plate shall not be less than 8 mm exclusive of corrosion allowance. Sometimes stainless steel cladded skin plate is used to get rid of corrosion.

The number of girders used shall depend on the total height of the gate, but shall be kept as minimum as possible. The spacing between main horizontal girders shall preferably be such that all the girders carry almost equal loads. The horizontal girders should also be suitably braced to ensure structural rigidity.

Invariably all welded girders with thickness of plates greater than 36 mm should be stress relieved.

Number of arms per side is equal to number of horizontal girders, unless vertical end girders are provided. The arms may be straight or parallel. Inclined arms are also conveniently used. The arms shall be suitably braced.

Figure 8-5: Radial Arms and Trunnion

Figure 8-7 Spillway Radial Gates

Figure 8-6: Radial Gate Trunnion

The arms of the gate shall be rigidly connected to the trunnion hubs to ensure full transfer of loads. The trunnion hubs shall rotate about the trunnion pins. The trunnion pin shall normally be supported at both ends on the trunnion bracket which is fixed to the anchorage or support girder. The trunnion pin shall be hard chrome plated, if it is not made of corrosion resistant steel.

The trunnions shall be so located that the resultant hydraulic thrust through the gate in the closed position with reservoir water level at F.R.L lies as close to the horizontal as possible. In case of conduits and tunnels, the trunnion shall be located clear of the water profile under free flow conditions.

Radius of the gate shall vary from H to 1.25 H, where H is the height of gate.

The trunnion anchorage comprises essentially of a trunnion / yoke girder, held to the concrete of the spillway piers by anchor rod or plate sections designed to resist the total water thrust on the gate.

The thrust may be distributed in the concrete either as bond stress along the length of the anchors or as a bearing stress through the medium of an embedded anchor girder. In the latter case, the anchors are insulated from the surrounding concrete. In case of large sized gates where very high loads are required to be transferred, pre-stressed anchorage system is used.

At least two side guide rollers on each side shall remain within the wall plate area when the gate is in fully raised condition. A minimum load of 5 % of the total dead weight of the gate is recommended for the design of guide rollers on either side.

The anchorages shall be provided in the first stage concrete, with suitable block out openings, to hold the embedded parts to be covered in the second stage concrete. The minimum size of anchor bolts shall not be less than 16 mm diameter.

The skin plate and other components of radial gate which may have a sustained contact with water are designed for wet condition. Girders, stiffeners etc. of radial gate which generally do not have a sustained contact with water are designed for dry condition. In case of gates likely to be overtopped, the end arms and other components should be suitably protected by provision of hood & or side shields to prevent direct impact of water on arms. A suitably shaped & sized hood may also be provided to protect the horizontal girders and other downstream parts from overflowing water. Provision of suitable sized flow splitters / breakers made of converging shapes be provided for aeration under the separating sheet of water.

The gate is checked for earthquake effect according to the seismic zone of the project. The gate is also checked for Maximum Water Level (MWL) condition. The allowable stresses are increased by 33.33 % of the values specified in IS: 4623.

8.4.4.3 Slide gate

Slide gate is vertical lift type gate and most commonly used for medium and high head. General design of the gates involves design of the following components:

a) Skin plate
b) Vertical / horizontal stiffeners and main horizontal girders
c) Bearing plate and sliding track
d) Seals & accessories and seal seat & seal base
e) Guide rollers / guide shoes and guides
f) Sill beam
g) Body, bonnet and bonnet cover (for bonneted type slide gate)
h) Anchors and Embedded parts (1st stage anchors & 2nd stage embedded parts)
i) Gate lifting attachments
j) Dogging beam / Latching arrangement

Gate leaf is designed as per IS: 5620 for low head slide gate and IS: 9349 for medium and high head slide gate. The thickness of the skin plate shall not be less than 8 mm exclusive of corrosion allowance.

Figure 8-8: Vertical Lift Slide Gate with Gantry crane and Lifting Beam

The end vertical girders are designed as continuous beam having concentrated loads from horizontal girders and uniform reaction from bearing plate.

Maximum deflection of the gate under normal conditions of loading shall be limited to L / 800 of the span (c/c of bearing plate) for low head slide gate (IS: 5620) and L/2000 of the span (c/c of bearing plate) for medium and high head slide gate. But in case of bulk head slide gate, maximum deflection of the gate shall be limited to L/1200 of the span as per IS: 9349.

The bearing plate / pad for the slide gates are usually made of brass / bronze or stainless steel. The bearing pad transmits the load to the bearing plate. The slide track transmits the heavy loads from the bearing pad / plate into the concrete pier without allowing the stress in concrete to exceed the permissible stress in pier.

The gate is made in a number of elements for ease of transportation, if required. The elements of gate are joined by splicing with rubber gaskets on either side of the skin plate for making the joint totally leak proof.

In case of medium and high head gates, the bottom shape of the gate shall be suitably designed to minimise down pull in case of gates with downstream sealing and to minimise uplift and vibrations in case of gates with upstream sealing. It should be designed to provide a converging water way and stream lined flow . Sometimes, stainless steel plate is overlaid on the bottom profile of gate to protect from corrosion. For aeration, provision of airvent of adequate size shall be kept.

For medium and high head under sluice or silt flushing gate, bonneted type slide gate is provided with body, bonnet and bonnet cover along with hydraulic hoist or screw hoist. The body and bonnet are embedded in concrete. These should be sufficiently reinforced to withstand the water pressure. Sufficient reinforcing ribs in horizontal and vertical direction are provided to prevent damage or distortion during transportation and installation. The bonnet parts are either with flange bolted joints at top and bottom, or in welded construction without flanged joints, maintaining strict tolerances for gaps around the gate. All flanged joints should be provided with O-ring rubber gasket.

Bonnet cover should be designed to withstand the full internal water pressure. In installations where hoist is directly mounted over the bonnet cover, it should be designed to resist the full load of maximum hoisting effort in addition to water pressure. Gland stuffing box should be provided on bonnet cover to prevent leakage of water around stem rod of gate leaf passing through the bonnet cover. Provision for venting of air should be made in the bonnet cover.

For gates with water on both sides of the skin plate, the maximum water thrust corresponds to the most unfavorable unbalanced level between the upstream and the downstream reservoirs is considered to design the gate. Sometimes, skin plates on both sides of gate are provided. Double bulb music note type side seal and top seal is used to prevent water from both sides.

The gate is checked for earthquake effect according to the seismic zone of the project. The gate is also checked for Maximum Water Level (MWL) condition. The allowable stresses are increased by 33.33 % of the values specified in IS: 5620 and IS: 9349.

The gate is normally designed to close under its own weight with or without addition of ballast, but a positive thrust for closing may be required for medium and high head slide gates.

8.4.4.4 Stop logs

Stop logs are provided at upstream of vertical gate or radial gate. Stop logs may be fixed wheel type or sliding type. These are made in number of elements or small units to economise on the hoist capacity. The stop logs facilitate the maintenance of main crest gates or other gates.

Design criteria of stop logs are same as fixed wheel vertical gates (IS: 4622) and slide gates (IS: 5620 & IS: 9349) as mentioned above.

The stop log units are generally lifted by gantry crane (IS: 3177) with a lifting beam (IS: 13591) in balanced head condition.

Generally all units of stop logs are made inter-changeable except top unit. Top unit is provided with filling-in valve to achieve balanced head condition.

Latching arrangement is used for latching of stop log units inside the grooves at deck level, when not in use.

During monsoon period, stop logs shall never be lowered in spite of heavy leakage through seals of main gate.

8.4.4.5 Trash racks and Trash Rack Cleaning Machine

a) Trash racks

Trash racks are provided at the entrance of intakes to protect turbines, pumps, valves etc. from objectionably large debris.

Trash racks are designed as per IS: 11388. General design of the trash racks involves design of the following components:

a) Horizontal members / beams
b) Vertical members / beams
c) Trash bars

Trash bars are designed for 6.0 m differential head. Other steel supporting members (horizontal and vertical members) are designed for 7.0 m differential head of water. Spacing of trash bars shall be as per Indian standard. Trash racks should be so designed that head loss is minimum.

Stability of trash racks against vibration is checked in accordance with Indian standard.

Trash rack panels should be made identical to simplify site erection.

Trash racks are generally cleaned by manually or mechanically by Trash Rack Cleaning Machine (TRCM).

For racks which are to be cleaned mechanical means, the slope of trash rack should be 10° to 15 ° with vertical.

The bars of any panel should be directly in line with the corresponding bar above or below, so that cleaning rake operates satisfactorily while passing up and down the screen.

Not more than 50% of the trash rack area should be allowed to clog the racks at any time. Design of trashrack and associate structure shall be such that the velocity through the trashracks will not be more than 0.75 m/sec (with manual cleaning) and 1.5 m/sec (with mechanical / machine cleaning provision). Trashrack shall also be checked for vibration. Trashracks located in very cold regions i.e where formation of ice around trashbars/trashrack are likely to occur, heating provision as per standard shall be kept.

b) Trash Rack Cleaning Machine (TRCM)

Trash Rack Cleaning Machine reliably remove deposited foreign bodies from the protective rack in front of the inlet opening and ensure unobstructed flow of water. Depending upon the scope of contamination, trash rack cleaning machine secure high energy use of hydro power station and guarantee an increase in capacity of up to 30%.

The efficiency of a hydro power plant is directly related to reduction of head loss. Therefore, proper trash rack cleaning during full operation of the plant is a top priority.

TRCM may be stationary or movable depending upon the length of rack to be cleaned. It may be rope operated or hydraulically operated depending upon the cleaning depth.

TRCM can be equipped with additional crane and auxiliary equipment like lifting grapplers for handling trash racks and removal of large size floating objects like trees, logs etc. at the upstream of trash racks.

As there is no separate Indian standard for TRCM, most of the mechanical parts of TRCM is designed by IS: 3177 and structural parts are designed by IS: 807 like gantry crane.

For rope operated TRCM, weight of rake bucket should be sufficient to go at the bottom of trash rack easily. Size of rake bucket depends on trash handling capacity of TRCM.

The hoisting and lowering of rake bucket is done by electrically operated rope drum

Figure 8-9: Trash Rack Cleaning Machine

hoist. But, the opening and closing of rake bucket is controlled by hydraulic actuator. Generally, lowering speed is provided more than lifting speed.

The trash is collected into the trash container by means of hydraulically operated movable chute or deflector plate. Trash is then disposed by trash trolley or truck.

The operations of TRCM, i.e. cleaning cycle may be semi-automatic or automatic. All operations are controlled from operator's cabin.

8.4.4.6 Rope Drum Hoist

Rope drum hoist is most commonly used for operation of self-lowering fixed wheel type vertical gates and radial gates. Rope drum hoist is economical for low to medium capacity in comparison to hydraulic hoist.

Hoist capacity of vertical lift gates / stop logs and radial gates is determined by taking into consideration dead weights plus the worst combination of all frictional forces. Hoist capacity thus arrived shall be increased by 20% as reserved capacity. Buoyancy force is neglected during hoist capacity calculation, but it is taken into consideration while checking for self-seating / self-lowering of gate.

Rope drum hoist is designed as per IS: 6938. General design of the rope drum hoist involves design/selection of the following mechanical/electrical and structural components:

a) Wire rope
b) Rope drum
c) Sheaves or pulleys
d) Gearing / gearboxes
e) Shafts and bearings
f) Hoist motor and limit switches
g) Brakes (electro-magnetic & Thruster type) and couplings
h) Gate position indicator
i) Manual operation arrangement.
j) Squirrel cage Induction Motors
k) Control Equipment
l) Earthing arrangement
m) Hoist support structure

Wire rope shall be normally 6x36 or 6x37 construction as per IS: 2266. Minimum factor of safety of wire rope should 6 for normal operation condition and 3 for breakdown torque condition of motor.

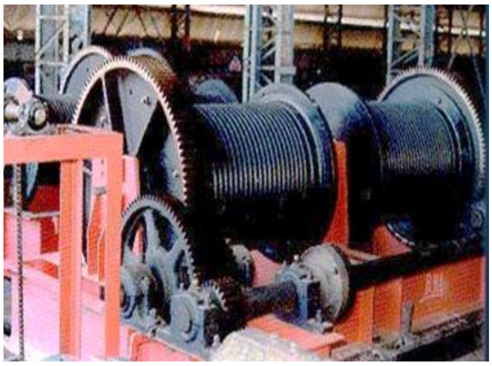

Figure 8-10: Rope Drum Hoist

Rope drum shall be machined groove and made of cast steel or mild steel. Sheaves or pulleys shall be machined groove and sheave guards shall be provided to retain the ropes in the groove.

Standard worm or helical gear boxes should be used for heavy reduction in addition to open gearing. Gear box should be preferably self-locking type.

Motor shall be totally enclosed and fan cooled. The break down torque of motor should be less than 2 times of the rated torque. Actual break down torque of selected motor should be used to check the hoist components in Break down Torque / Pull out Torque condition. In addition to provision of motor overload relay, mechanical arrangement for protection of rope overload and slack rope adjuster should be provided.

Hydraulic thruster brake or any additional brake is provided in the hoist system in addition to electro-magnetic brake to arrest undesirable gravity fall of gate where the selected gearbox is not of self-locking type.

Dial type gate position indicator is provided in the hoist system to know the position of gate during lifting or lowering cycle. Digital type position indicator may also be provided.

Manual operation is provided for emergency operation of gate in the event of electric supply failure. It should be so designed that not more than four number of persons are required for its operation.

Complete rope drum hoist assembly is generally load tested at 125% of over load preferably at manufacturer's work shop.

8.4.4.7 Gantry Crane and Lifting beam

Gantry crane with a lifting beam is most commonly used for handling of stop logs / vertical gates for isolation of downstream installations.

General design of the gantry crane involves design of the following mechanical/electrical and structural components:

a) Wire rope
b) Rope drum
c) Sheaves or pulleys
d) Gearing / gearboxes
e) Shafts and bearings
f) Motors and limit switches
g) Brakes and couplings
h) Control Equipment
i) Trolley frame
j) Main girders and cross girders
k) Gantry legs and Tie beams
l) Ladders and walkway
m) Operators cabin
n) Wheels / wheel bogie assemblies for LT drive, LT Rail arrangement with end stoppers.
o) Pulley block
p) Self-engaging & disengaging type lifting beam assembly.
q) Motorised / Spring operated / Counter- weight operated cable reeling drum.
r) Cross travel drive with rails & end stoppers.

Gantry crane is designed for suitable class of crane (generally M5 i.e. class II) as per IS: 3177 (Mechanical part) and IS: 807 (Structural part).

Wire rope working under water and in corrosive environment / atmosphere should be galvanised.

Minimum hardness of wheel rim of gantry crane should be maintained 300 to 350 BHN with hardness depth of10 mm (min.). Wheel adhesion shall be checked as per IS: 3177 to eliminate slipping of the driving wheels of long travel mechanism.

The stability of gantry crane in different conditions of loading is very important and should be checked as per relevant Indian standard. Span (c/c between rails) and wheel base (c/c between outer wheels) should be as large as possible for proper stability of gantry crane. Counter weight is to be provided for stability on the gantry crane, if required.

Generally a Lifting Beam of self-engaging and disengaging type as per IS: 13591 is used with gantry crane for lifting / lowering of gates and stop logs / bulkheads.

Figure 8-11: Gantry Crane with Lifting Beam

Lifting Beam frame shall mainly comprise of two number structural steel channels or fabricated channels with back to back connection to make it a single fabricated structural frame.

Two side guide rollers /shoes shall be provided on each side of the lifting beam. The depth of lifting beam / frame should be sufficient to accommodate two rollers on either side located at sufficient vertical distance from one another to enable proper guided movement. The depth of lifting beam shall not be less than one tenth of the length / span of the lifting beam or 500 mm whichever is more.

Lifting beam hook mechanism shall provide for automatic engagement and release of the equipment to be handled manually by movement of the hook block. The two hooks shall be mechanically linked together for simultaneous operation.

A probe release mechanism shall be provided with lifting beam along with probe rod in the stop log unit, so that lifting hook will disengage only when stop log unit rests on the sill beam / another stop log unit.

Complete gantry crane along with lifting beam assembly is generally load tested at 125% of over load preferably at manufacturer's work shop.

8.4.4.8 Hydraulic Hoist

Hydraulic hoist is most commonly used for operation of radial gates and vertical lift slide gates / fixed wheel gates. Hydraulic hoist has greater mechanical efficiency and ease of speed & control. It can work under water and can be mounted in number of positions, but height of lift of hydraulic hoist has limitations on account of buckling phenomena during compressive loading (L / R ratio) on its stem as well as due to limitations of its manufacturing facilities.

A hydraulic hoist consists of a cylinder with upper and lower cylinder head, piston and stem passing through a packing in the lower cylinder head. The hoists are operated by a motor and oil pump arrangement with the directional control valves which are actuated by electric contacts to any desired position.

The following factors generally govern the choice of hydraulic hoists:

a) High capacity and low travel
b) Large range of hoisting / lowering speed
c) Limited space availability
d) Dampening of vibration of gates, and
e) Requirement of positive thrust
f) Lesser maintenance

General design of the hydraulic hoist involves design of the following components:

a) Cylinder and Cylinder heads
b) Piston and Piston stem / rod
c) Couplings
d) Piston rings, seals and packings
e) Gate position indicator
f) Hydraulic power pack including oil tank, piping etc.
g) Electrical control equipment

Hydraulic hoist is designed as per IS: 10210. A maximum design pressure of 200 kg/cm^2 (200 bar) is considered in the design. The hoist components are tested at 150 % of the operating pressure for a period not less than 30 minutes.

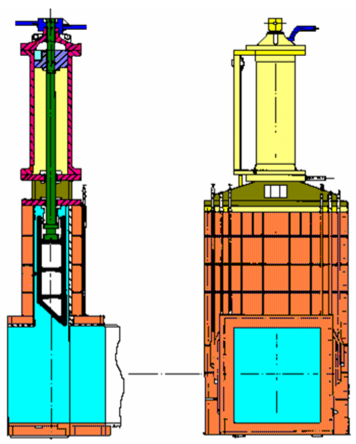

Figure 8-12: Slide gate with Hydraulic Hoist

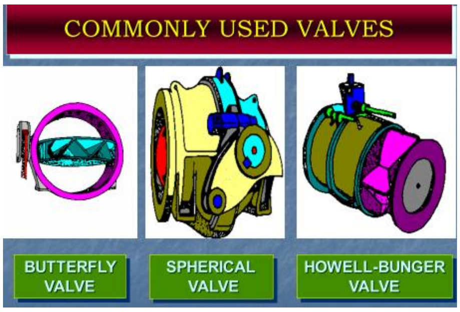

Figure 8-13: Valves for Hydro-Mechanical Works

Single acting hydraulic hoist is used for gravity closing gates. But, when positive thrust is required to close the gate, double acting hydraulic hoist is used.

Piston stem is generally made of stainless steel. Piston stem of carbon steel with hard chrome plating is also used to protect from corrosion. Piston stem is checked in buckling during design.

Towards the end of the closing stroke, it is desirable to slow down the speed of the gate to have a dampening effect.

Two motor driven oil pumps should be provided for the operating system to ensure the operation of 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.

Hand pump is also provided with hydraulic system to operate the gate manually.

The hydraulic hoist system is so designed that in case the gate jams and it is beyond the capacity of the hydraulic hoist to lift it, pressure will rise to the setting of the relief valve and the pump oil will return to tank without overloading the hoist system.

Position indicator is provided with hydraulic hoist to indicate position of gate.

Now a days for the Gates operated with Hydraulic hoists, oil contamination checking kit with oil purification and low vaccum de-hydration and degasification provision are also being kept in projects.

8.4.4.9 Screw Hoist

Screw hoist is generally used for the operation of small size low head vertical canal gates as per IS: 11228. It may be manually operated or motorised type. It is very compact and more economical as compared to other types of hoists. The screw hoist is used when a positive thrust is required to close the gate.

General design of the screw hoist involves design of the following components:

a) Stem and Nut
b) Gearing and gear boxes
c) Keys and couplings
d) Pedestal
e) Manual operation
f) Bonnet and Bonnet cover
g) Gate position indicator
h) Electrical equipment

Screw hoists of capacity up to 7.5 tonnes may normally be provided with only manual operation arrangement. However, screw hoists of larger capacities should be provided with electrical operation along with an emergency hand operation arrangement.

8.4.4.10 Commonly used Valves

Butterfly valve, spherical valve and Howell-Bunger valve are commonly used in hydro-mechanical installations to control flow of water.

The valves may be operated manually by hand wheel or electrically by hydraulic actuator. The body and seat of valves should be pressure tested with water as per relevant standard preferably at manufacturer's workshop. The valves are also checked for any leakage through body / seat.

Regular operation and maintenance of valves are recommended for trouble free service and long life of valves.

8.4.5 Rehabilitation and Replacement works on old and aged hydro-mechanical installations

8.4.5.1 General

Rehabilitation / replacement works involving renovation, strengthening of aged, weakened components of gates embedded parts & surrounding concrete structures, replacement of out-lived components of hoists / hoist platforms & supporting steel / concrete structures are required to be handled on old and aged gate installations.

The hydro-mechanical equipment's are an essential part of a dam project. These equipment provides the necessary control for the dam project. Whereas, the civil works can be expected to have a useful life of well over 100 years with regular & proper maintenance, the hydro-mechanical installations along with associated electrical components usually do not last more than about 30 or 40 years with regular maintenance. Thereafter, the rehabilitation, renovation and replacement of damaged /defective / deteriorated component plays more important role for the balance useful life of the project. The problems experienced by hydro-mechanical equipment that limit its life are caused by the joint effects of regular wear & tear, corrosion, erosion and poor and irregular maintenance. The key to prolonged trouble free operational life, is to follow sustained and well planned routine services along with the occasional break-down interventions, as per requirements to be assessed by regular detailed inspections including under water inspections & repair works.

8.4.5.2 Rehabilitation of Gates and Other Discharge Equipment's

Replacement/rehabilitation activities on gate installations are required as per the condition monitoring requirements without any delay as there are no hard & fast yard sticks for the same.

The principles of inspections, maintenance, upkeep and rehabilitation, basic concerns in respect of trained manpower, regular preventive maintenance & rehabilitation with proper budgeting approvals, have already been covered in great details in the "Manual for rehabilitation of large dams" which needs to be referred to and acted upon in letter & spirit.

8.4.5.3 Integrity checks on old and aged gate installations

The inspections for condition monitoring and remedial rehabilitation works require detailed non-destructive testing of all important and critical components of the gates, hoists, embedded parts , checking the actual dimensions, surface defects, internal defects in structural members, surface cracks, reduction in thickness of structural members, damages in associated concrete works . NDT tests comprising Dye penetrant tests , Radiographic test / X-Ray , Ultrasonic Test (UT) on structural steel for surface cracks, internal defects , weld defects etc.. These tests help in working out the remedial action plan for rehabilitation and replacement of components, which needs to be attended for renovation of the weakened components to bring them as close as possible to their original strengths as originally supplied and installed by the original H M equipment manufacturers.

The investigations / testing requirements for the Integrity Review Protocol to be adopted prior to undertaking the rehabilitation works is covered in detail in already published manual named: "Manual for Rehabilitation of large dams" under clause 7.3.3 titled "Investigations / testing before rehabilitation works - Integrity Review Protocol Requirements", which may be referred to.

8.4.5.4 Issues with small size vertical lift type sluice gates

The old and aged dams built long ago generally had provision of small sized sluice gates & hoists. Such gates are required to be operated very regularly at a given frequency to remain functional & healthy.

However due to irregular operations, maintenance, majority of such old & aged sluice gates which remained non-functional were abandoned & simply forgotten. This situation is prevailing at many of the Indian aged dams. Such installations have the potential to cause catastrophic accidents endangering the safety & security of the dam structure and the manpower. It therefore becomes advisable to plug such installations permanently.

In recent times, the present day designers in India prefer to go for provision of low level medium to high head sluice radial type gates with deep crest to serve as low level easily operable and maintainable radial gates. Such gate installations remain usefully functional over the life of dam without causing functional hassles faced by small sized vertical lift type sluice gates.

Following are some of the dams with small sized sluice gates / valves installed for sluicing purposes but have remained non-functional over a long period of time. Some of the glaring examples are brought out below:

(a) DVC Dams

i. Maithon Dam

There are 5 No. under sluices in the concrete dam portion of Maithon dam with 5 numbers under sluice gates of high pressure slide type of size 1.73 m (wide) x 3.05 m (high). Discharge capacity of each sluice is 113 cumec. There is one under sluice emergency gate which is made of single flat leaf, vertical lift, fixed wheel type to close an opening of 2.85 m wide by 4.56 m high. One number Gantry crane of 40 T capacity mounted on rail is used to lift under sluice emergency gate from its dogged position and close the under sluices for servicing of under sluice service gates. The emergency and service sluice gates were non-operative since 2008 at Maithon Dam.

ii. Panchet Dam

There are 10 No. under sluices in the concrete dam portion of Panchet dam with ten number under sluice gates (9 nos. high pressure slide type) of size 1.73 m (wide) x 3.05 m (high) and one Harza gate (butterfly type) to close an opening of the same size as above with discharge capacity of 98 cumec each. One under sluice emergency gate of single flat leaf, vertical lift, fixed wheel type is provided to close an opening of 2.85 m wide by 4.56 m high. One gantry crane of 40 T capacity mounted on rail is used for lifting under sluice emergency gate and putting them in the under sluice slots to close under sluices for repair and maintenance work. All the nine sluice gates were stated to be non-operational since 2008 (8 in fully closed position & 1 stuck in 100 mm open position), while the Harza gate (butterfly type- patented) was never operated since commissioning at Panchet Dam. Heavy leakages were noticed from the stuffing boxes of few of the hydraulic hoists and were flooding the hoist gallery for which dewatering pumps were installed.

iii. Konar Dam:

There are 20 no. under sluices in the concrete dam portion of Konar Dam of 2.29 m diameter each. Vertical lift electrically operated. Gats are provided. Discharge capacity of each sluice is 95 cumec. One number under sluice emergency gate of size 3.5 m x 3.87 m high, single flat leaf, vertical lift, fixed wheel type has been provided. One number Gantry crane of 28 T capacity mounted on rail is used to lift under sluice emergency gate from its dogged position and close the under sluices for servicing of under sluice service gate. This is also used for lifting stop logs for crest spillway gates. The under sluice emergency gates were found to be non-functional, sluice gates were able to close fully, the common gantry crane for emergency sluice gates and spillway stop logs were non-functional for last 25 years at Konar Dam.

iv. Proposed investigations & rehabilitation works for DVC dams:

Inspection and videography of underwater critical components of under-sluice emergency gates is to be got done for Maithon, Panchet & Konar dams to identify the scope of work & rehabilitation of the underwater components.

The rehabilitation works for Gantry cranes, lifting beams & emergency sluice gates for isolation of under sluices and inspection for identification of the scope of work of embedded parts, liners, bonnet and bonnet covers, hydraulic hoists and power packs is to be got conducted prior to taking up their rehabilitation works.

After successful commissioning of the sluice emergency gates and dewatering the sluices, the embedded parts / liners, bonnet, bonnet covers (for Maithon, Panchet, & Konar) including manholes, air vents need to be inspected and rehabilitated. The existing sets of the hydraulic hoists are to be rehabilitated and provided with new modular power packs (for Maithon and Panchet) and rehabilitation of motorised screw hoists for Konar.

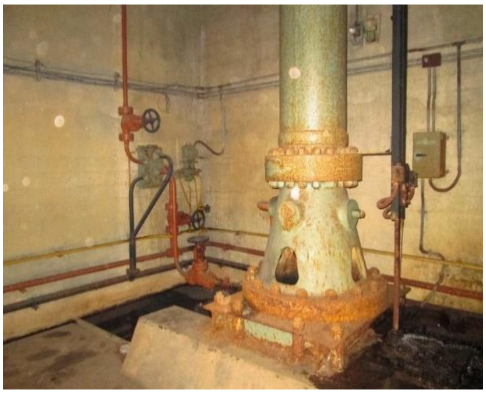

Figure 8-14: Maithon - Under sluice gate, bonnet cover and hydraulic hoist

Figure 8-15: Maithon - Power pack installations for under sluice gate

Figure 8-16: Panchet - Profuse leakage was observed from the stuffing box of the under sluice gate bonnet covers in few of the sluice gates, causing flooding of the sluice hoist gallery.

Figure 8-17: Konar – Water leaking from the under sluices

Figure 8-18: Konar – Non-functional hoisting system of gantry crane

Figure 8-19: Under sluice gates and hoist installation

Figure 8-20: Panchet – Upstream Under sluice emergency gate in dogged position

Figure 8-21: Konar – Incomplete, damaged electrical components

(b) MPWRD Projects

i. Tawa Dam

The project was completed in 1974. The under sluice gates had remained non-functional since 1983. The gates were designed & installed by M/S Tungabhadra Steel Products Ltd.

There are four under sluices provided in the dam body of sizes 1.83 m (W) x 2.43 m (H). All these sluices are provided with one number sluice service gate of vertical fixed wheel type and one number sluice emergency gate in a steel lined body with bonnet & bonnet covers. A rectangular manhole cover is also provided for access for inspection / service & repair purposes. In addition, one upstream stop log provision with a lifting beam and operated through gantry crane has been kept for isolation of the emergency & service sluice gates with screw stem operated hoists.

The emergency and service gates for the sluices comprising of liners, bonnet and bonnet covers and hoist system remain inoperative since 1985 without any attempt for rehabilitation.

It was intimated that the stem of the screw hoist of sluice gate no. 2 had got bent during the past and had been got replaced.

Physically, the bonnet & bonnet cover and manhole structures appear to be structurally in good condition as per the visual inspection, except that no servicing, upkeep, painting etc. was done since 1983.

The bonnet and bonnet covers remain sealed by about fifty-five years' old rubber seals and fasteners and therefore the same were checked for replacement to avoid sudden leakage due to decomposition. The stop log and its lifting beam and gantry crane were recommended for rehabilitation for isolation of the sluice gates for making it functional.

ii. Barna Dam:

Two number sluices comprise of two number 600 mm dia. pipes each of which is manned independently by a manually operated sluice valves. As per drawing, two valves for each sluice were shown, the u/s acting as an emergency valve for isolation to be kept open, while the d/s valve serving as service valve. However, at the site only one number valve for each of the two sluices are provided. Both these valves are in a dilapidated condition, badly rusted & non-functional since a long time. The reason for flowing water found around the second valve needs to be investigated as it may be from the u/s pipe or the seepage through the civil structure. Dilapidated sluice valves (not in operation since installation), U/S stop logs gantry crane frame without a hoist and LT drive, stop log (without lifting beam) are lying abandoned.

Barna Dam (MP) got commissioned in 1975. The manually driven sluice valve with piping system remained inoperative since inception without any attempt for rehabilitation at a later stage. It is a dangerous move as the valve is badly rusted for more than 41 years. The body of the valve / rusted pipe may burst any time without warning. The project has never tried to rehabilitate the upstream stop log and its gantry crane lying as junk and the stop log upstream area must have got silted.

iii. Proposed investigation & rehabilitation works for the above MPWRD dams:

For rehabilitation and /or replacement of the sluice gates / valves, the sluices shall require to be isolated one by one by lowering of the stop logs in each sluice by gantry crane.

However, it will require a complete overhauling & rehabilitation or replacement of the un-used & non-functional stop logs & lifting beams, rehabilitation / replacement of the fully dilapidated gantry structure & its manual operating mechanism by a new gantry crane with electro-mechanical hoist mechanism/installation for operation.

The stop log needs to be rehabilitated & modified to make the stop logs as per IS 5620 considering additional silt load up to the top of stop log in addition to maximum water load. The gantry crane & lifting beam / stop log for Barna dam also needs to be replaced.

The silt level in the reservoir u/s of the sluices is required to be got assessed by sounding or by satellite imaging prior to lowering the sluice stop logs. For rough estimation of u/s silt, a metal piece load tied with the rope was lowered in front of the sluices during the site visit. It was assessed that silt level is still below the bottom of the sluice as per measurement of the rope done by the project engineers.

The sluice stop logs need to be placed in position with the help of divers (if these do not get engaged with guides provided on the d/s face of the dam starting from much below the spillway crest) to isolate the reservoir prior to checking the sluice valves, piping along its layout & its structure for damages & to establish the leakage locations near the valve no. 2 (which may be from the u/s piping or civil structure) for necessary remedial measures.

To replace the single manual valve on each pipe which appear to have outlived their service life by provision of two sets of u/s & d/s motor actuated knife edge gate valves spaced at comfortable distance for maintenance, with the u/s one to work as emergency valve for repairs / maintenance of the d/s valve and d/s valve as service valve.

The valves need to be operated at a regular interval based on the silt build up in the reservoir to ensure their effective repetitive operation and not left in closed position for long periods of time.

The silt level against the sluices needs to be lowered and not allowed to build up beyond the safe limit and needs to be monitored regularly.

Figure 8-22: Tawa – Under sluice gate and hoist installation

Figure 8-23: Tawa – Under sluice gate and hoist installation (Enlarged view)

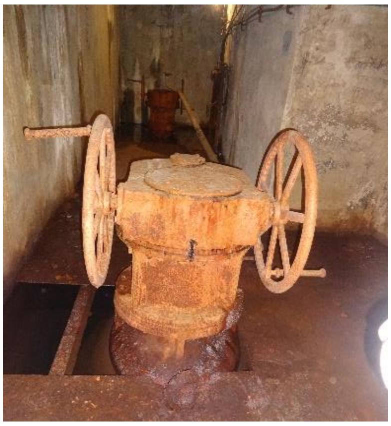

Figure 8-24: Barna – Dilapidated sluice valve (not in operation since installation)

Figure 8-25: Barna – U/S stop logs gantry crane frame without a hoist and LT drive, stop log (without lifting beam) lying in abandoned.

8.4.5.5 Conclusion

The sustenance of sluice gate installations for a longer useful service life, demands a regular operation and maintenance schedule to be followed religiously and timely undertaking renovation/ replacement and up gradation of damaged components of structures and their control equipment's.

The availability of the trained manpower with practical exposure and experience, O&M Manuals and "As built" drawings & document and regular annual maintenance work plans with budgetary support, open access to new technologies, persistent follow up, result oriented management are essential requirements for keeping this equipment healthy and workable over the life of the dam. Inaction by the projects for years together is a very dangerous trend as it could lead to a catastrophic accident. The bonnet, bonnet covers, manholes and their covers can become a source of sudden flooding of the sluice gallery due to the disintegration of seals, weakened fasteners, and stuffing boxes as the bonnet and bonnet covers remain pressurized by reservoir water level while the strength and integrity of various component parts remain unknown.

8.4.6 Dam Owner's Responsibility:

The dam owners have a great responsibility to look after the operation, maintenance, upkeep, rehabilitation / renovation / modernization requirements of the dams at various stages which are the basic assets of the Nation for the welfare of the whole population and above all the safety of the downstream population.

Dam owners, therefore, have a great responsibility on their shoulders. At various stages of the life of dam, there are bound to be changes in the organizational structure, manpower, resources, skills etc. The head of the organization in consultation with his team needs to take up the critical issues periodically with State Government as required for the safety & security of the dam installations, to tide over the problems or shortages, if any, to take appropriate care of the criticalities etc. This is a very important & critical function for the dam owners at highest level of responsibilities.

Some of the basic critical concerns are as under:

- Non-availability or deficit of trained manpower at site, design departments etc.
- Financial restrictions on budgets for maintenance & rehabilitation.
- Irregular & unplanned system of servicing, maintenance.
- Non-availability of design, drawings, O&M manuals at site.
- Non-availability of maintenance closure provisions for gates.
- Contractors with inadequate resources and facilities at sites.
- In-effective and irregular inspections, inadequate preventive maintenance at sites.

8.4.7 Maintenance protocol for gates and hoists

ROUTINE MAINTENANCE CHECKS:

i) General cleanliness of embedded parts, gates & hoist components.

ii) Check for oil level in worm reducer / helical gearbox.

iii) Lubrication of pulleys and pins by grease gun / hand on trunnion pin bush bearing, guide rollers, gate wheel bearings, rope drum bush bearing, gear wheels, spur gear bearings, line shaft bearings, manual operation mechanism and other related parts. Iv)

iv) Movement of wheels and guide rollers should be smooth and it can be rotated by hand.

v) Check condition of seals.

vi) Check for operation of brakes.

vii) Check for loose electrical connections.

YEARLY MAINTENANCE CHECKS:

i) Check the tightness of foundation bolts of motors, worm reducers, plummer blocks and coupling joints.

ii) Check for smooth operation of gate by raising and lowering (without noise, jerks or vibration).

iii) Check the operation of gantry crane with lifting beam and opening & closing of filler valve of stop log top unit.

iv) Check for condition of painting of all components.

v) Check the dryness of wire rope before every monsoon, clean and apply cardium compound.

vi) Check clogging of drain holes including in end arms, horizontal girders etc.

MAINTENANCE CHECKS EVERY THREE YEARS

i) Check the condition of wirerope, pulleys, sheaves, limit switch, brakes and gear wheels.

ii) Check gate seals for damages (crack, wear and tear).

iii) Check seal bolts for damages.

MAINTENANCE CHECKS EVERY SIX YEARS:

i) Check integrity of load bearing members, welds & joints for damages at skin plate joints, tee girders to horizontal girders, horizontal girder to arm, arm bracings, horizontal girder bracings, end boxes, gate stiffeners, hoist-bridge etc.

ii) Check wheel assemblies for any breakage, freezing , corrosion and misalignment.

iii) Check sill beam, side guides, and roller tracks for damages, corrosion and pitting requiring strengthening /replacement.

iv) Check the hoist-bridge foundation bolts for tightening.

8.4.6 List of Indian Standards for Hydro-Mechanical Works

Some of the Bureau of Indian Standard (BIS) codes needed for the Hydro-Mechanical works are given in the list of references at the end of this Manual

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IS: 10096 (P-2) – Recommendations for Inspection, Testing and Maintenance of Radial gates and Rope drum hoists: At the time of Erection

IS: 10096 (P-3) – Recommendations for Inspection, Testing and Maintenance of Radial gates and Rope drum hoists: After Erection

IS: 13053 – Commissioning and Maintenance of complete hydraulic systems – Recommendation

IS: 13623 – Criteria for choice of Gates and Hoists

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APPENDIX A – STABILITY ANALYSIS CALCULATIONS OF NON-OVER FLOW AND OVER FLOW SECTIONS

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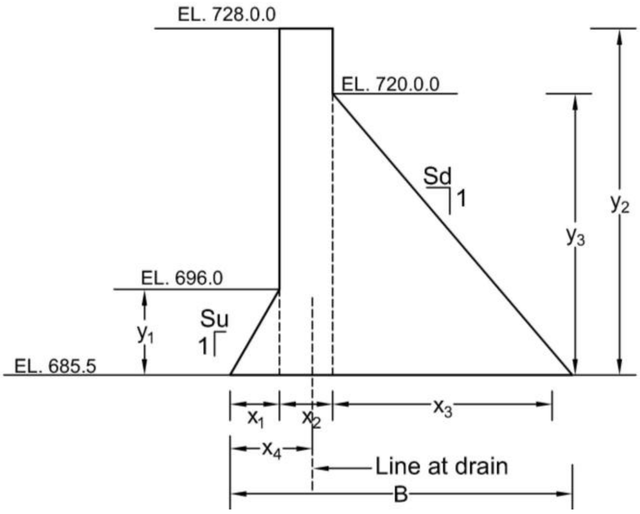
Gravity Dam Stability Analysis (NOF-Block)

The stability analysis of Non-overflow concrete dam has been carried out as per procedure given in IS 6512 -1984 and IS 1893-1984. (*For earthquake loading conditions this type of analysis is recommended for preliminary analysis for dams of height upto 15 m only*)

Basic Data

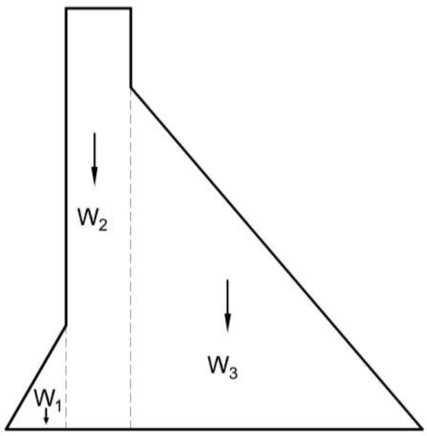
Top of the dam Level	=	EL.728.0
Full reservoir level	=	EL.725.0
Maximum water level	=	EL.725.0
MDDL	=	EL.721.0
Silt Level	=	EL.696.0
Maximum Tail water level	=	EL.685.5
Minimum tail water level	=	EL.685.5
Foundation Level	=	EL.685.5
Total Height of the dam	=	42.5
Head at FRL (h_1)	=	39.5
Head at MWL (h_2)	=	39.5
Top width	=	6.5
U/S slope (Su)	=	0.10
D/S slope (Sd)	=	0.85
Elevation from which U/S slope starts	=	EL.696.0
Elevation from which D/S slope starts	=	EL.720.00
Distance of line of drains from u/s heal (x_4)	=	6.05
Min. Tail water depth corresponding to FRL (h_3)	=	0.0
Max. Tail water depth corresponding to MWL (h_4)	=	0.0
Compressive strength of concrete fc	=	1500
Design horizontal seismic coefficient (α_h)	=	0.24
Design vertical seismic coefficient (α_v)	=	0.16
Unit weight of concrete (γ_c)	=	2.4
Unit weight of water (γ_w)	=	1
Vertical density of silt (γ_{sv})	=	0.925
Horizontal density of silt (γ_{sh})	=	0.36
Shear parameters at dam foundation interface	=	
c	=	50
tan Φ	=	0.7

Here,			
x_1	=	1.05	m
x_2	=	6.50	m
x_3	=	29.33	m
y_1	=	10.50	m
y_2	=	42.50	m
y_3	=	34.50	m
Base width (B) = $x_1+x_2+x_3$= 36.88 M			



Calculation of Forces

1. Self-Weight of the dam



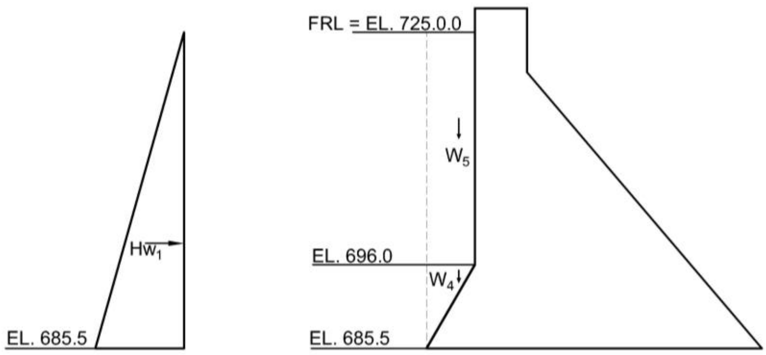
Sl. No.	Particulars	Vertical Force (t)	Lever arm (m)		Moment (t-m)
1	$W_1=1/2*x_1*y_1*\gamma_c$	13.23	$x_1*2/3$	0.70	9.26
2	$W_2=x_2*y_2*\gamma_c$	663.00	$x_1+x_2/2$	4.30	2850.90
3	$W_3=1/2*x_3*y_3*\gamma_c$	1214.06	$x_1+x_2+x_3/3$	17.33	21033.50
	Total	1890.29			23893.66

2. U/S Water Pressure

(a) Reservoir at FRL

(i) Horizontal Water Pressure

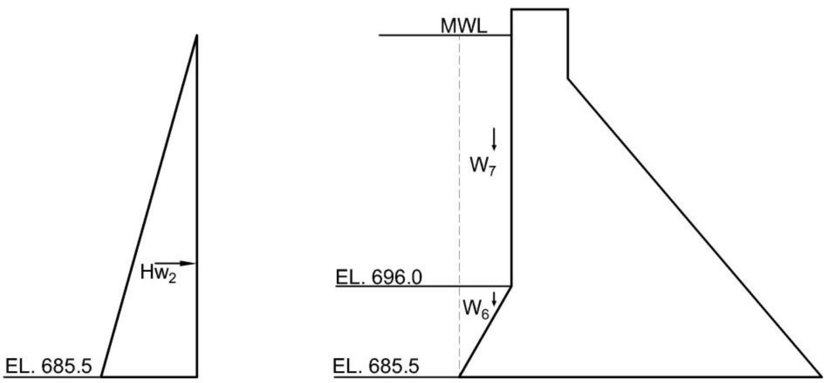
	$Hw_1 = 1/2*h_1^{2}*\gamma w$	780.125	t
	Lever arm = $h_1/3$	13.17	m
	Moment	10271.65	t-m



(ii) Vertical Weight of Water

Sl. No.	Particulars	Vertical Force (t)	Lever arm (m)	Moment (t-m)
1	$W_4=1/2*x_1*y_1*\gamma_w$	5.51	$x_1/3 =$ 0.35	1.93
2	$W_5=x_1*(h_1-y_3)*\gamma_w$	30.45	$x_1/2 =$ 0.53	15.99
	Total	35.96		17.92

(b) Reservoir at MWL



(i) Horizontal Water Pressure

	$Hw_2 = 1/2*h_2^2*\gamma w$	780.125	t
	Lever arm = $h_2/3$	13.17	m
	Moment	10271.65	t-m

(ii) Vertical Weight of Water

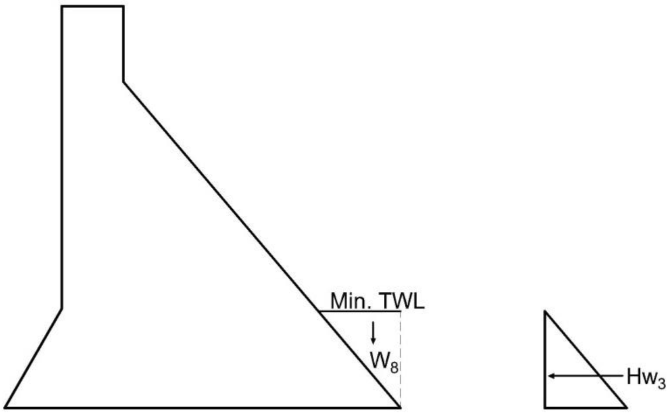
Sl. No.	Particulars	Vertical Force (t)	Lever arm (m)	Moment (t-m)
1	$W_6=1/2*x_1*y_1*\gamma_w$	5.51	$x_1/3 =$ 0.35	1.93
2	$W_7=x_1*(h_2-y_3)*\gamma_w$	30.45	$x_1/2 =$ 0.53	15.99
	Total	35.96		17.92

3. Tail Water Pressure

(a) Minimum TWL

(i) Horizontal Tail Water Pressure

	$Hw_3 = 1/2*h_3^2*\gamma_w$	0	T
	Lever arm = $h_3/3$	0.00	M
	Moment	0.00	t-m



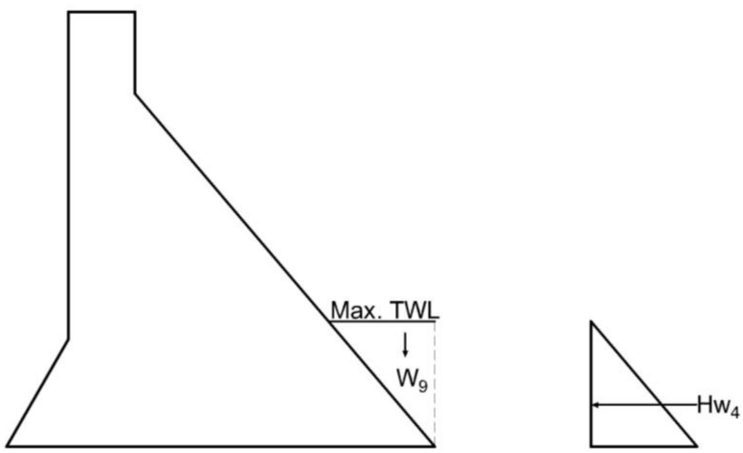
(ii) Vertical Weight of Tail Water

Sl. No.	Particulars	Vertical Force (t)	Lever arm (m)		Moment (t-m)
1	$W_8=1/2*S_d*h_3*h_3*\gamma_w$	0.00	$x_1+x_2+(x_3-S_d*h_3/3)=$	36.88	0.00
	Total	0.00			0.00

(b) Maximum TWL

(i) Horizontal Tail Water Pressure

$Hw_4 = 1/2*h_4^2*\gamma_w$	0	t
Lever arm = $h_4/3$	0.00	m
Moment	0.00	t-m

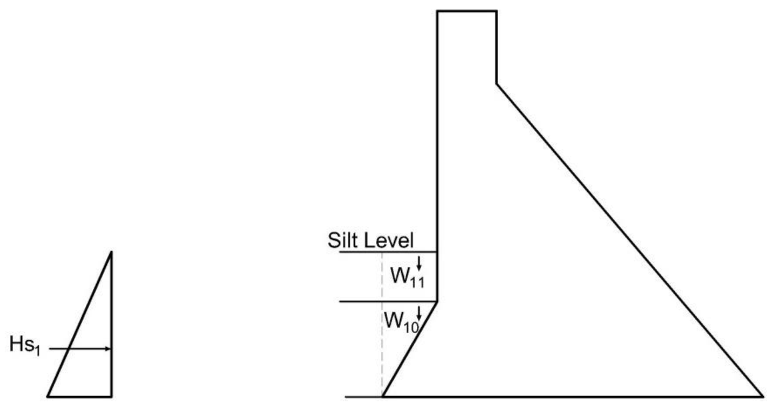


(ii) Vertical Weight of Tail Water

Sl. No.	Particulars	Vertical Force (t)	Lever arm (m)	Moment (t-m)
1	$W_9=1/2*S_d*h_4*h_4*\gamma_w$	0.00	$x_1+x_2+(x_3-S_d*h_4/3)=36.88$	0.00
	Total	0.00		0.00

4. **Silt pressure**

Depth of silt (hs) = 10.5 m



(i) Horizontal Silt Pressure

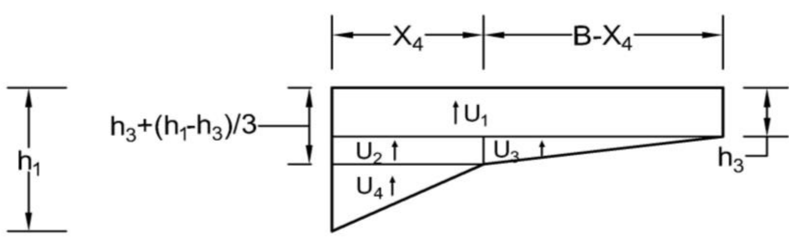
	$Hs= 1/2*hs^2*\gamma_{sh}$ =	19.845	t
	Lever arm = hs/3 =	3.50	m
	Moment =	69.46	t-m

(ii) Vertical Weight of Silt

Sl. No.	Particulars	Vertical Force (t)	Lever arm (m)	Moment (t-m)
1	W_{10}= if(hs<=y_1,$1/2*hs^2*Su*\gamma_{sv}$, $1/2*x_1*y_1*\gamma_{sv}$)	5.10	hs*Su/3= 0.35	1.78
2	W_{11}=if(hs<=y_1,0, $x_1*(hs-y_1)*\gamma_{sv}$)	0.00	$x_1/2$= 0.53	0.00
	Total	**5.10**		**1.78**

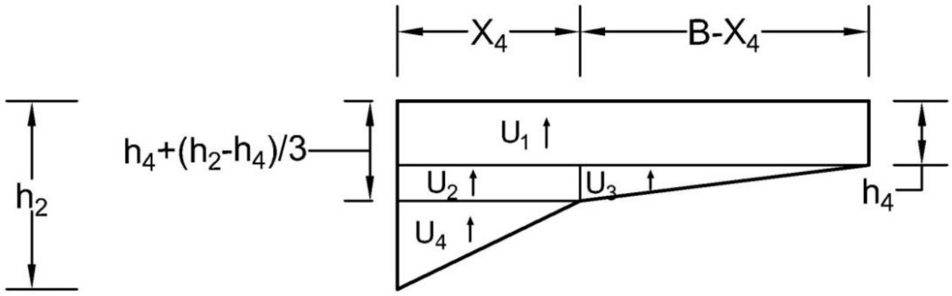
5. **Uplift pressure (Normal)**

(a) FRL



Sl. No.	Particulars	Vertical Force (t)	Lever arm (m)		Moment (t-m)
1	U_1=B*h_3*γ_w	0.00	B/2=	18.44	0.00
2	U_2=x_4*(h_1-h_3)/3*γ_w	79.66	x_4/2=	3.03	240.97
3	U_3=1/2*(B-x_4)*(h_1-h_3)/3 *γ_w	202.93	x_4+(B-x_4)/3=	16.33	3312.85
4	U_4=1/2*x_4*(h_1-h_3-(h_1-h_3)/3) *γ_w	79.66	x_4/3=	2.02	160.64
	Total	**362.25**			**3714.46**

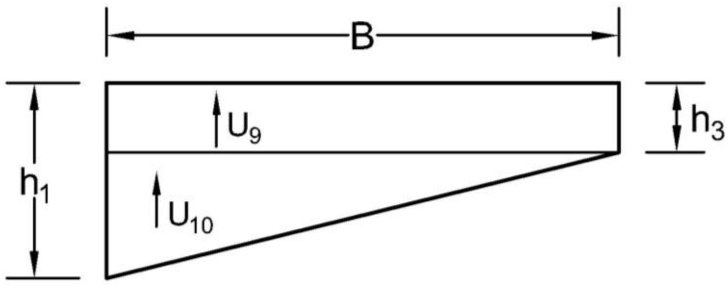
(b) MWL



Sl. No.	Particulars	Vertical Force (t)	Lever arm (m)		Moment (t-m)
1	U_5=B*h_4*γ_w	0.00	B/2 =	18.44	0.00
2	U_6=x_4*(h_2-h_4)/3*γ_w	79.66	x_4/2 =	3.03	240.97
3	U_7=1/2*(B-x_4)*(h_2-h_4)/3 *γ_w	202.93	x_4+(B-x_4)/3 =	16.33	3312.85
4	U_8=1/2*x_4*(h_2-h_4-(h_2-h_4)/3)*γ_w	79.66	x_4/3 =	2.02	160.64
	Total	**362.25**			**3714.46**

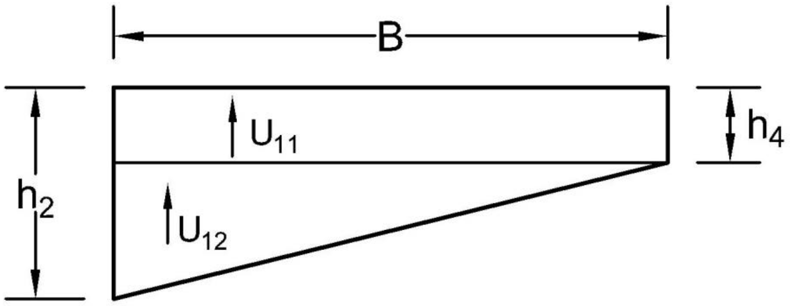
6. **Uplift pressure (Extreme)**

   (a) FRL



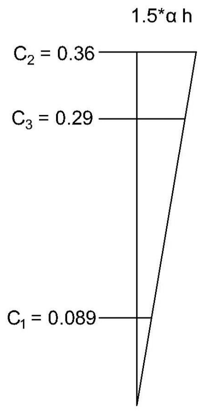
Sl. No.	Particulars	Vertical Force (t)	Lever arm (m)		Moment (t-m)
1	U_9=B*h_3*γ_w	0.00	B/2 =	18.44	0.00
2	U_{10}=B*(h_1-h_3)/2*γ_w	728.28	B/3 =	12.29	8951.79
	Total	**728.28**			**8951.79**

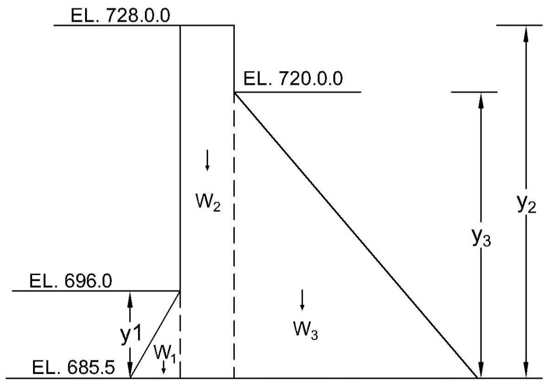
(b) MWL



Sl. No.	Particulars	Vertical Force (t)	Lever arm (m)		Moment (t-m)
1	U_{11}=B*h_4*γ_w	0.00	B/2 =	18.44	0.00
2	U_{12}=B*(h_2-h_4)/2 *γ_w	728.28	B/3 =	12.29	8951.79
	Total	**728.28**			**8951.79**

7. Horizontal Inertia Forces/ Earthquake



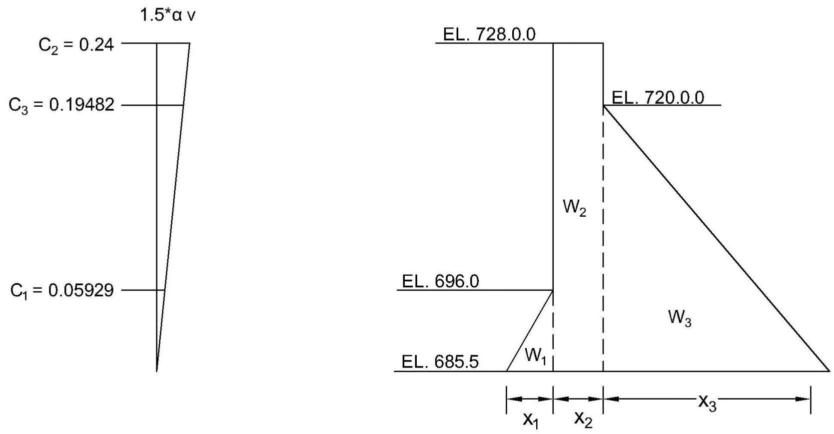


Horizontal Inertia Force

Sl. No.	Particulars	Horizontal Force (t)	Lever arm (m)		Moment (t-m)
1	On W_1=W_1*C_1/3	0.39	y_1/2 =	5.25	2.06
2	On W_2=W_2*C_2/2	119.34	2*y_2/3 =	28.33	3381.30
3	On W_3=W_3*C_3/3	118.26	y_3/2 =	17.25	2040.04
	Total	**238.00**			**5423.40**

8. Vertical Inertia Forces / Earthquake

Vertical Inertia Forces



Sl. No.	Particulars	Vertical Force (t)	Lever arm (m)		Moment (t-m)
1	On W_1=W_1*C_1/3	0.26	3*x_1/4 =	0.79	0.206
2	On W_2=W_2*C_2/2	79.56	1/2*x_2+x_1 =	4.30	342.108
3	On W_3=W_3*C_3/3	78.84	x_1+x_2+x_3/4 =	14.88	1173.270
	Total	**158.66**			**1515.58**

9. Hydrodynamic Pressure Due to Earthquake

Hydrodynamic Pressure (p_e) =	Cs*α_h*γ_w*h_1 =	6.9678	t/m^2
	h_1 =	39.5	m
C_m =	0.735		
Cs =	C_m/2[y/h(2-y/h)+ sqrt{y/h(2-y/h)}]		0.735
At foundation level, y=h_1=		39.5	m
Hydrodynamic force=V_v =	0.726*p_e*y =	199.82	t
Hydro-dynamic Moment =My =	0.299*p_e*y^2=	3250.58	t-m

Load Combinations

1. Load Combination A: (Reservoir Empty)

Sl. No.	Particulars	Vertical Force (t)	Lever arm (m)	Moment (t-m)
1	Self-Weight of the dam	1890.29		23893.66
	Total	**1890.29**	**12.64**	**23893.66**

x = (ΣM/ΣW) =	12.64	m
e= (B/2-x) =	5.80	m
6e/B =	0.94	
Stress at u/s= ΣW(1+6e/B)/B =	99.62	t/m^2
Stress at d/s= ΣW(1-6e/B)/B =	2.91	t/m^2

2. **Load Combination B: (Reservoir at FRL, Normal Uplift, without Earthquake, Minimum TWL)**

Sl. No.	Particulars	Vertical Force (t)	Horizontal Force (t)	Lever arm (m)	Moment (t-m)
1	Self-Weight of the dam	1890.29			23893.66
2	Horizontal water pressure		780.125		10271.65
3	Weight of water	35.96			17.92
4	Horizontal silt pressure		19.845		69.46
5	Weight of Silt	5.10			1.78
6	Tail Water Pressure		0		0.00
7	Weight of tail water	0.00			0.00
8	Uplift pressure (Normal)	-362.25			-3714.46
	Total	**1569.10**	**799.97**	**19.46**	**30540.00**

x = (ΣM/ΣW) =	19.46	m
e= (B/2-x) =	-1.03	m
6e/B =	-0.17	
Stress at u/s= ΣW(1+6e/B)/B	35.45	t/m^2
Stress at d/s= ΣW(1-6e/B)/B	49.65	t/m^2

$F = \left(\sum \frac{(w-u)\tan\phi}{F\phi} + \frac{CA}{Fc} \right) \div H$	=	1.56

F= factor of safety against sliding	
tan ø = Coefficient of internal friction of the material	0.7002
C= Cohesion at contact between rock and concrete	50
A= area under consideration for cohesion	36.88
Fø= partial factor of safety in respect of friction	1.5
Fc=partial factor of safety in respect of cohesion	3.6

3. **Load Combination C: (Reservoir at MWL, Normal Uplift, without Earthquake, Max. TWL)**

Sl. No.	Particulars	Vertical Force (t)	Horizontal Force (t)	Lever arm (m)	Moment (t-m)
1	Self-Weight of the dam	1890.29			23893.66
2	Horizontal water pressure		780.13		10271.65
3	Weight of water	35.96			17.92
4	Horizontal silt pressure		19.85		69.46
5	Weight of Silt	5.10			1.78
6	Tail Water Pressure		0.00		0.00
7	Weight of tail water	0.00			0.00
8	Uplift pressure (Normal)	-362.25			-3714.46
	Total	**1569.10**	**799.97**	**19.46**	**30540.00**

x = (ΣM/ΣW) =	19.46	m	
e= (B/2-x) =	-1.03	m	
6e/B =	-0.17		
Stress at u/s= ΣW(1+6e/B)/B		35.45	t/m^2
Stress at d/s= ΣW(1-6e/B)/B		49.65	t/m^2

$F = \left(\sum \dfrac{(w-u)\tan\phi}{F\phi} + \dfrac{CA}{Fc} \right) \div H$	=	1.56

F= factor of safety against sliding	
tan ø = Coefficient of internal friction of the material	0.7002
C= Cohesion at contact between rock and concrete	50
A= area under consideration for cohesion	36.88
Fø= partial factor of safety in respect of friction	1.5
Fc= partial factor of safety in respect of cohesion	3.6

4. Load Combination D: (Combination A, with Earthquake)

Sl. No.	Particulars	Vertical Force (t)	Horizontal Force (t)	Lever arm (m)	Moment (t-m)
1	Load combination A	1890.29			23893.664
2	Horizontal Inertia Force		-238.00		-5423.40
3	Vertical Inertia Force	158.66			1515.58
	Total	**2048.95**	**-238.00**	**9.75**	**19985.85**

x = (ΣM/ΣW) =	9.75	m	
e= (B/2-x) =	8.68	m	
6e/B =	1.41		
Stress at u/s= ΣW(1+6e/B)/B		134.07	t/m^2
Stress at d/s= ΣW(1-6e/B)/B		-22.94	t/m^2

$F = \left(\sum \dfrac{(w-u)\tan\phi}{F\phi} + \dfrac{CA}{Fc} \right) \div H$	=	8.25

F= factor of safety against sliding	
tan ø = Coefficient of internal friction of the material	0.7002
C= Cohesion at contact between rock and concrete	50
A= area under consideration for cohesion	36.88
Fø= partial factor of safety in respect of friction	1.2
Fc= partial factor of safety in respect of cohesion	2.4

5. Load Combination E: (Combination B, with Earthquake)

Sl. No.	Particulars	Vertical Force (t)	Horizontal Force (t)	Lever arm (m)	Moment (t-m)
1	Load combination B	1569.10	799.97		30540.00
2	Horizontal Inertia Force		238.00		5423.40
3	Vertical Inertia Force	-158.66			-1515.58
4	Hydrodynamic Pressure		199.82		3250.58
	Total	**1410.43**	**1237.78**	**26.73**	**37698.40**

C	26.73	m
e= (B/2-x) =	-8.29	m
6e/B=	-1.35	
Stress at u/s= ΣW(1+6e/B)/B	-13.35	t/m^2
Stress at d/s= ΣW(1-6e/B)/B	89.85	t/m^2

$F = \left(\sum \frac{(w-u)\tan\phi}{F\phi} + \frac{CA}{Fc} \right) \div H$	=	1.29

F= factor of safety against sliding	
tan ø = Coefficient of internal friction of the material	0.7002
C= Cohesion at contact between rock and concrete	50
A= area under consideration for cohesion	36.88
Fø= partial factor of safety in respect of friction	1.2
Fc= partial factor of safety in respect of cohesion	2.4

6. Load Combination F: (Combination C, but Drains Choked)

Sl. No.	Particulars	Vertical Force (t)	Horizontal Force (t)	Lever arm (m)	Moment (t-m)
1	Load Combination C	1569.10	799.97		30540.00
2	Less Normal Uplift	362.25			3714.46
3	Add Extreme Uplift	-728.28			-8951.79
	Total	**1203.07**	**799.97**	**21.03**	**25302.68**

x = (ΣM/ΣW) =	21.03	m
e= (B/2-x) =	-2.59	m
6e/B =	-0.42	
Stress at u/s= ΣW(1+6e/B)/B	18.85	t/m^2
Stress at d/s= ΣW(1-6e/B)/B	46.40	t/m^2

$F = \left(\sum \frac{(w-u)\tan\phi}{F\phi} + \frac{CA}{Fc} \right) \div H$	=	2.97

F= factor of safety against sliding	
tan ø = Coefficient of internal friction of the material	0.7002
C= Cohesion at contact between rock and concrete	50
A= area under consideration for cohesion	36.88
Fø= partial factor of safety in respect of friction	1
Fc= partial factor of safety in respect of cohesion	1.2

7. Load Combination G: (Combination E, But Drains Choked)

Sl No	Particulars	Vertical Force (t)	Horizontal Force (t)	Lever arm (m)	Moment (t-m)
1	Load Combination E	1410.43	1237.78		37698.40
2	Less Normal Uplift	362.25			3714.46
3	Add Extreme Uplift	-728.28			-8951.79
	Total	**1044.40**	**1237.78**	**31.08**	**32461.07**

x = (ΣM/ΣW) =	31.08	m
e= (B/2-x) =	-12.64	m
6e/B=	-2.06	
Stress at u/s= ΣW(1+6e/B)/B	-29.94	t/m^2
Stress at d/s= ΣW(1-6e/B)/B	86.59	t/m^2

$F = \left(\sum \frac{(w-u)\tan ø}{Fø} + \frac{CA}{Fc} \right) \div H$	=	1.83

F= factor of safety against sliding	
tan ø = Coefficient of internal friction of the material	0.7002
C= Cohesion at contact between rock and concrete	50
A= area under consideration for cohesion	36.88
Fø= partial factor of safety in respect of friction	1.0
Fc= partial factor of safety in respect of cohesion	1.2

Summary of Results for NOF Section (At El. 685.50 m)

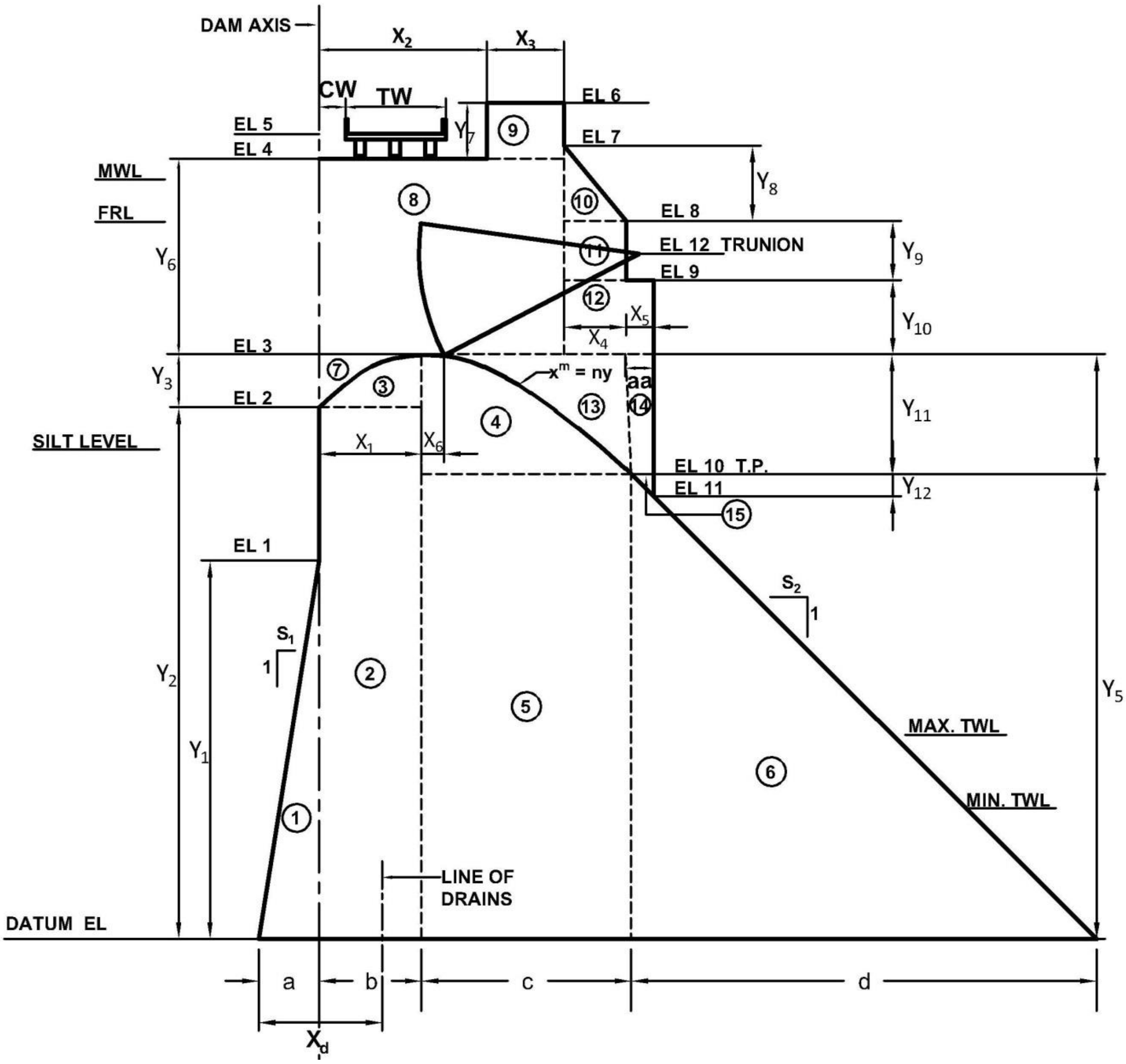
Load combina-tion	Stress U/s (t/m^2)	Stress D/s (t/m^2)	Factor of safety	Allowable Tensile Stress (t/m^2)
A	99.616	2.907		-
B	35.449	49.655	1.56	No Tension
C	35.449	49.655	1.56	15
D	134.071	-22.941	8.25	
E	-13.349	89.847	1.29	30
F	18.853	46.398	2.97	30
G	-29.944	86.590	1.83	60

Gravity dam - Stability Analysis (OF-Block)

The stability analysis of Overflow section has been carried out as per IS 6512 -1984 and IS 1893-1984. (*For earthquake loading conditions this type of analysis is recommended for preliminary analysis for dams of height upto 15 m only*)

Basic Data:

Top of the dam Level (Bridge top)	EL.585.0	m
Top of the pier Level	EL.588.5	m
Full reservoir level	EL.581.0	m
Maximum water level	EL.583.0	m
MDDL	EL.563.250	m
Silt Level	EL.563.250	m
Spillway Crest Level	EL.569.0	m
Maximum Tail water level	EL.560.2	m
Minimum tail water level	EL.541.0	m
Foundation Level (Datum El.)	EL.541.0	m
Trunnion Level	EL.573.0	m
Gate Size	12.0m (W) X13.0m (H)	
Clearence of spillway bridge from dam axis (CW)	1.80	m
Bridge Width (TW)	5	m
Bridge Weight/ m - run (BRWT)	20	t/m
Maximum Height (h)	47.5	m
Head at FRL (h_1)	40.0	m
Head at MWL (h_2)	42.0	m
Width of overflow block (BW)	16.0	m
Width of pier (PW)	4.0	m
Min. Tail water depth corresponding to FRL (h_3)	0.0	m
Max. Tail water depth corresponding to MWL (h_4)	19.2	m
U/S slope (S1)	0.050	
D/S slope (S2)	0.850	
Compressive strength of concrete fc	1500	t/sq.m
Design horizontal seismic coefficient (α_h)	0.06	g
Design vertical seismic coefficient (α_v)	0.03	g
Unit weight of spillway crest section material (γ_{sp})	2.4	t/cu.m
Unit Weight of spillway pier material (γ_p)	2.4	t/cu.m
Unit weight of water (γ_w)	1	t/cu.m
Vertical submerged density of silt (γ_{sv})	0.925	t/cu.m
Horizontal submerged density of silt (γ_{sh})	0.36	t/cu.m
Shear parameters at dam foundation interface		
c=	50	t/m^2
tan Φ=	0.7	



Here,

m = 1.85

n = 16.6666

Note: m & n represent constants of equation of the downstream quadrant of spillway crest (refer spillway section above)

EL.1 =	560.00	m	
EL.2 =	567.41	m	
EL.3 =	569.00	m	
EL.4 =	583.00	m	
EL.5 =	585.00	m	
EL.6 =	588.50	m	
EL.7 =	583.00	m	
EL.8 =	574.50	m	
EL.9 =	572.00	m	

EL.10 =	558.3670	m	
EL.11 =	557.77	m	
EL.12 =	573.00	m	
x_1 =	3.4060	m	
x_2 =	7.80	m	
x_3 =	4.50	m	
x_4 =	5.50	m	
x_5 =	3.00	m	
x_6 =	1.00	m	
x_d =	4.50	m (Distance of line of drains from U/S heel)	
y_1 =	19.00	m	(EL.1 - DATUM EL.)
y_2 =	26.41	m	(EL.2 - DATUM EL.)
y_3 =	1.59	m	(EL.3 - EL.2)
y_4 =	10.63	m	(EL.3 - EL.10)
y_5 =	17.37	m	(EL.10 - DATUM EL.)
y_6 =	14.00	m	(EL.4 - EL.3)
y_7 =	5.50	m	(EL.6 - EL.4)
y_8 =	8.50	m	(EL.7 - EL.8)
y_9 =	2.50	m	(EL.8 - EL.9)
y_{10}=	3.00	m	(EL.9 - EL.3)
y_{11} =	10.63	m	(EL.3 - EL.10)
y_{12} =	0.60	m	(EL.10 - EL.11)
a =	0.95		(y_1*S_1)
b=	3.41		(x1)
c=	16.42		((y_4*n)^(1/m))
d=	14.76		(y_5 * s_2)
aa=	0.97		(x_2+x_3+x_4+x_5)-(b+c)
Base width (B) = a+b+c+d =	35.54	m	

Notes:

(a) ***All forces are calculated per metre run.***

(b) ***Moments are about u/s heel.***

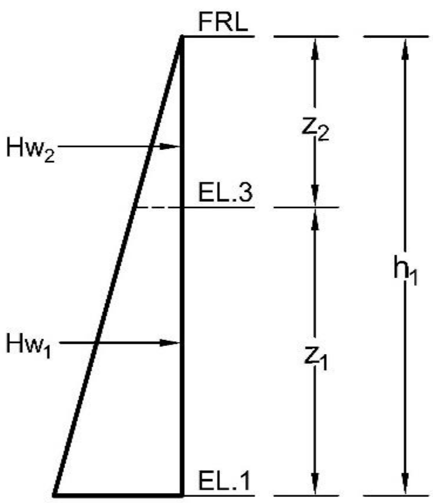
Calculation of Forces

1. Self-Weight of the dam:

Sl. No.	Particulars	Vertical Force (t)	Lever arm (m)		Moment (t-m)
Spillway Section					
1	$W_1=1/2*a*y_1*\gamma_{sp}=$	21.66	2a/3=	0.63	13.72
2	$W_2=b*y_2*\gamma_{sp}=$	215.89	a+b/2=	2.65	572.77
3	$W_3=2/3*b*y_3*\gamma_{sp}=$	8.66	a+2b/3=	3.22	27.89
4	$W_4=2/3*c*y_4*\gamma_{sp}=$	279.37	a+b+c/3=	9.83	2746.16
5	$W_5=c*y_5*\gamma_{sp}=$	684.45	a+b+c/2=	12.57	8601.26
6	$W_6=1/2*d*y_5*\gamma_{sp}=$	307.64	a+b+c+d/3	25.70	7905.84
Pier Section					
7	$W_7=(1/3)*b*y_3*\gamma_p*PW/BW=$	1.08	a+b/3 =	2.09	2.26
8	$W_8=(x_2+x_3)*y_6*\gamma_p*(PW/BW)=$	103.32	$a+(x_2+x_3)/2$	7.10	733.57
9	$W_9=x_3*y_7*\gamma_p*(PW/BW)=$	14.85	$a+x_2+x_3/2$	11.00	163.35
10	$W_{10}=(1/2)*x_4*y_8*\gamma_p*(PW/BW)=$	14.03	$a+x_2+x_3+x_4/3=$	15.08	211.54
11	$W_{11}=x_4*y_9*\gamma_p*(PW/BW)=$	8.25	$a+x_2+x_3+x_4/2=$	16.00	132.00
12	$W_{12}=(x_4+x_5)*y_{10}*\gamma_p*(PW/BW)=$	15.30	$a+x_2+x_3+(x_4+x_5)/2=$	17.50	267.75
13	$W_{13}=1/3*c*y_{11}*\gamma_p*(PW/BW)=$	34.92	a+b+2c/3 =	15.30	534.42
14	$W_{14}=aa*y_{11}*\gamma_p*(PW/BW)=$	6.21	a+b+c+aa/2 =	21.26	131.96
15	$W_{15}=1/2*aa*y_{12}*\gamma_p*(PW/BW)=$	0.17	a+b+c+2aa/3=	21.43	3.75
Spillway Bridge					
16	Self-Wt. of Bridge = BRWT	20.00	a+CW+TW/2 =	5.25	105.00
	Total	**1735.81**			**22153.23**

2. Water Pressure

(a) Reservoir at FRL

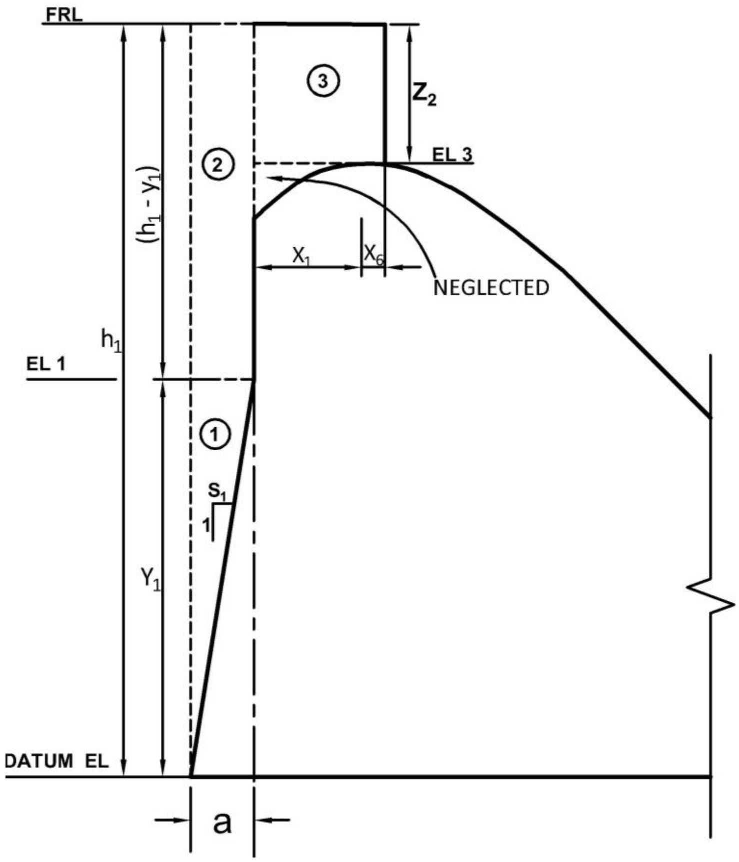


(i) Horizontal Water Pressure

h_1 =	40.0	m
Z_1 = EL.3-DATUM EL. =	28.00	m
$Z_2 = h_1 - Z_1$ =	12.00	m

Sl. No.	Particulars	Vertical Force (t)	Lever arm (m)		Moment (t-m)
1	H_{w1} (On Crest structure)				
	Triangular Portion = $1/2*Z_1^{2}*\gamma_w$ =	392.00	$Z_1/3$=	9.33	3658.67
	Rectangular Portion =$Z_2*Z_1*\gamma_w$ =	336.00	$Z_1/2$=	14.00	4704.00
2	H_{w2} (On Gate)				
	Triangular Portion = $1/2*z_2^{2}*\gamma_w*$(BW-PW)/BW =	54.00	EL.12-DATUM EL.	32.00	1728.00
3	H_{w3} (On Piers)				
	Triangular Portion = $1/2*z_2^{2}*\gamma_w*$(PW/BW) =	18.00	$Z_1+Z_2/3$=	32.00	576.00
	Total	**800.00**			**10666.67**

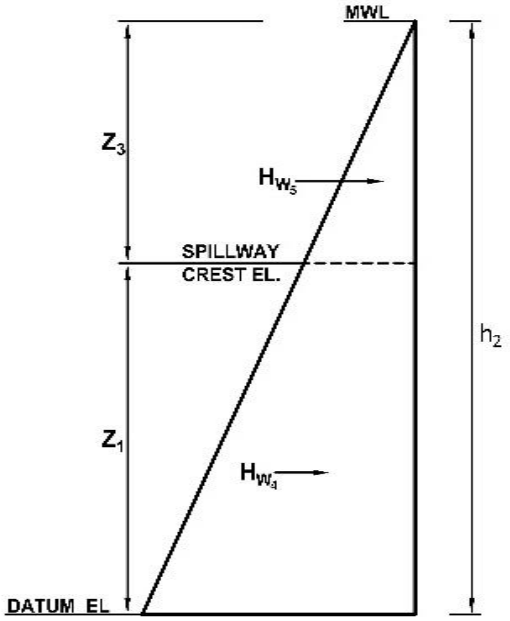
(ii) Vertical Weight of Water



Sl. No.	Particulars	Vertical Force (t)	Lever arm (m)		Moment (t-m)
1	Area 1 = $1/2*a*y_1*\gamma_w$ =	9.03	a/3=	0.32	2.86
2	Area 2 = $(h_1-y_1)*a*\gamma_w$ =	19.95	a/2=	0.48	9.48
3	Area 3 = $(x_1+x_6)*Z_2*((BW-PW)/BW)*\gamma_w$ =	39.65	$a+(x_1 + x_6)/2$ =	3.15	125.03
	Total	**68.63**			**137.36**

(b) Reservoir at MWL

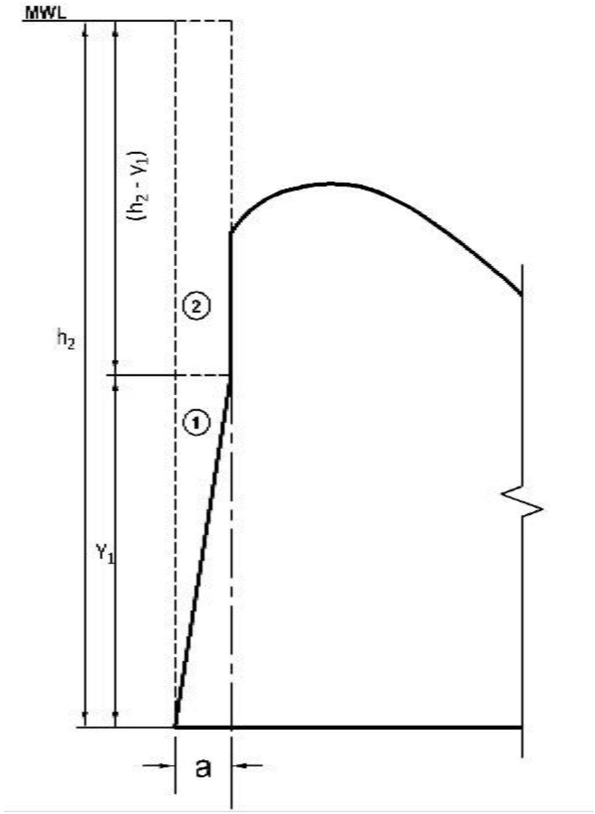
Horizontal Water Pressure



h_2 =	42.0	M
Z_1=	28.00	m
$Z_3 = h_2 - Z_1$ =	14.00	m

Sl. No.	Particulars	Vertical Force (t)	Lever arm (m)	Moment (t-m)
1	H_{w4}			
	Triangular Portion = $1/2*Z_1^2*\gamma_w$ =	392.00	$Z_1/3$= 9.33	3658.67
	Rectangular Portion =$Z_1*Z_3*\gamma_w$ =	392.00	$Z_1/2$= 14.00	5488.00
2	H_{w5} (on Piers)			
	Triangular Portion $1/2*z_3^2*\gamma_w$*PW/BW =	24.50	$Z_1+Z_3/3$= 32.67	800.33
	Total	**808.50**		**9947.00**

(i) Vertical Weight of Water



Sl. No.	Particulars	Vertical Force (t)	Lever arm (m)	Moment (t-m)
1	Area 1 = $1/2*a*y_1*\gamma_w$ =	9.03	a/3= 0.32	0.00
2	Area 2 = $a*(h_2-y_1)*\gamma_w$ =	21.85	a/2= 0.48	0.00
	Total	**30.88**		**0.00**

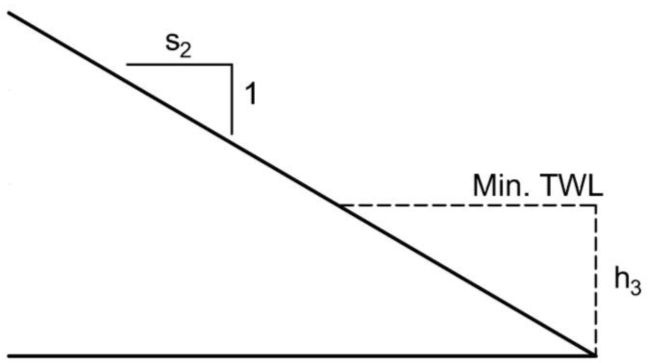
3. Tail Water Pressure

(a) Minimum TWL

(i) Horizontal Tail Water Pressure

$Hw_6 = 1/2*h_3^2*\gamma_w$ =	0	t
Lever arm = $h_3/3$ =	0.00	m
Moment =	0.00	t-m

(ii) Vertical Weight of Tail Water



Downstream slope =	S_2	0.850	
Minimum TWL =		EL.541.0	m
Foundation level = DATUM EL. =		EL.541.0	m
Minimum Tailwater depth corresponding to FRL	h_3	0.0	m

Sl. No.	Particulars	Vertical Force (t)	Lever arm (m)	Moment (t-m)
1	$1/2 \times h_3 \times h_3 \times S_2 \times \gamma_w$	0.00	$B - (h_3 \times S_2) / 3 = 35.54$	0
	Total	**0.00**		**0.00**

(b) Maximum TWL

(i) Horizontal Tail Water Pressure

$Hw_7 = 1/2 * h_4^2 * \gamma_w =$	183.9362	t
Lever arm = $h_4/3$ =	6.39	m
Moment =	1175.97	t-m

(ii) Vertical Weight of Tail Water

As water would be flowing this is neglected on safer side

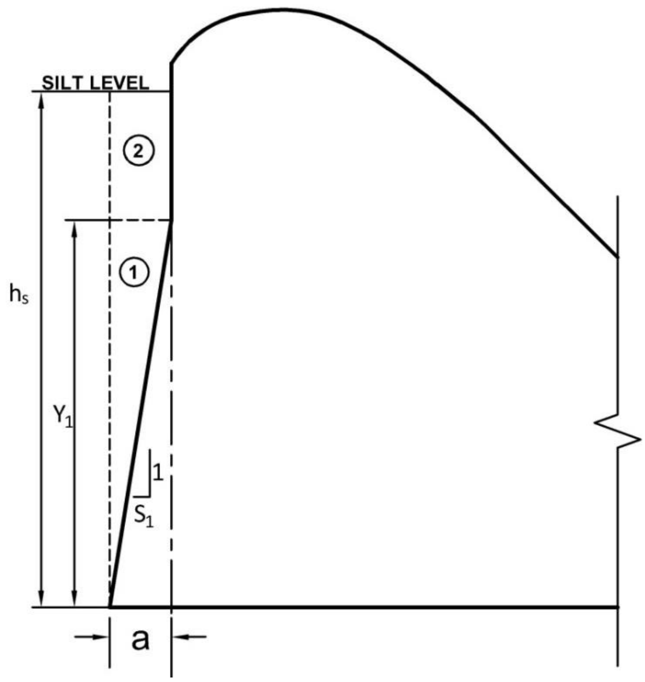
4. Silt Pressure

Depth of silt (hs)=	22.3	m

(i) Horizontal Silt Pressure

Silt Pressure = $1/2 * hs^2 * \gamma_{sh}$ =	89.11125	t
Lever arm = hs/3 =	7.42	m
Moment =	660.91	t-m

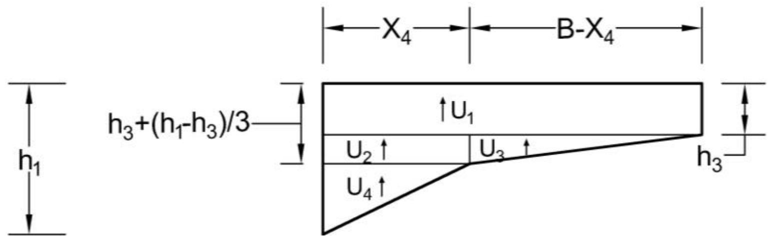
(ii) Vertical Weight of Silt



Sl. No.	Particulars	Vertical Force (t)	Lever arm (m)	Moment (t-m)
1	Area 1 = if(h_s<=y_1,1/2*h_s^2*S_1*γ_{sv}, 1/2*a*y_1* γ_{sv})	8.35	If (h_s<=y_1, h_s*S_1/3, a/3) = 0.32	0.00
2	Area 2 = if(h_s<=y_1,0, a*(h_s-y_1)* γ_{sv})	2.86	a/2 = .048	0.00
	Total	**11.20**		**0.00**

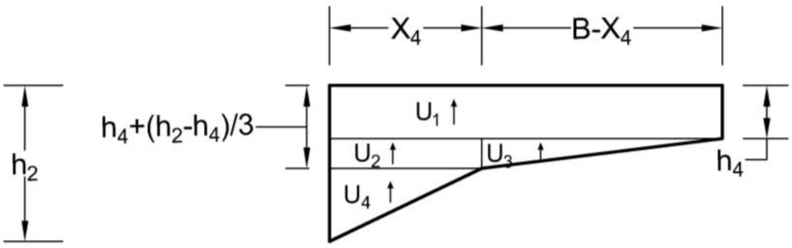
5. Uplift Pressure (Normal)

(a) FRL



Sl. No.	Particulars	Vertical Force (t)	Lever arm (m)		Moment (t-m)
1	U_1=B*h_3*γ_w =	0.00	=B/2=	17.77	0.00
2	U_2=x_d*(h_1-h_3)/3 *γ_w=	60.00	=x_d/2=	2.25	135.00
3	U_3=1/2*(B-x_d)*(h_1-h_3)/3 *γ_w=	206.93	=x_d+(B-x_d)/3=	14.85	3072.14
4	U_4=1/2*x_d*(h_1-h_3-(h_1-h_3)/3) *γ_w=	60.00	=x_d/3=	1.50	90.00
	Total	**326.93**			**3297.14**

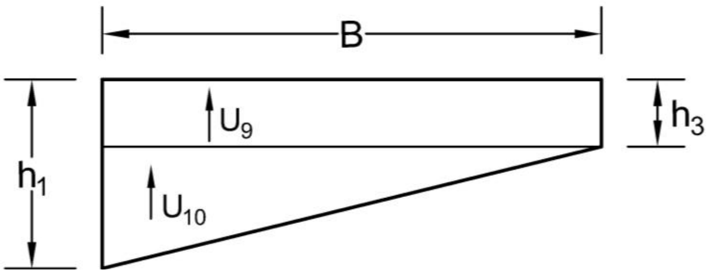
(b) MWL



Sl. No.	Particulars	Vertical Force (t)	Lever arm (m)	Moment (t-m)
1	$U_5=B*h_4*\gamma_w=$	681.64	B/2= 17.77	12112.52
2	$U_6=x_d*(h_2-h_4)/3*\gamma_w=$	34.23	$x_d/2$= 2.25	77.02
3	$U_7=1/2*(B-x_d)*(h_2-h_4)/3*\gamma_w=$	118.05	$x_d+(B-x_d)/3$= 14.85	1752.66
4	$U_8=1/2*x_d*(h_2-h_4-(h_2-h_4)/3)*\gamma_w=$	34.23	$x_d/3$= 1.50	51.35
	Total	**868.15**		**13993.54**

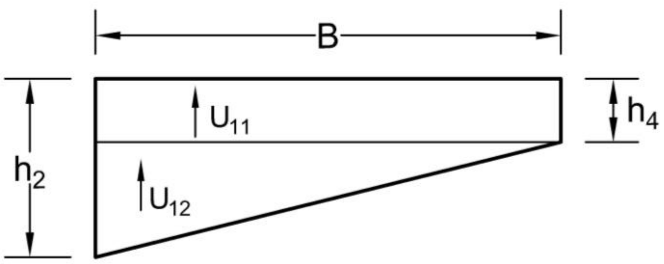
6. Uplift Pressure (Extreme)

(a) FRL



Sl. No.	Particulars	Vertical Force (t)	Lever arm (m)	Moment (t-m)
1	$U_9=B*h_3*\gamma_w=$	0.00	B/2= 17.77	0.00
2	$U_{10}=B*(h_1-h_3)/2*\gamma_w=$	710.78	B/3= 11.85	8420.24
	Total	**710.78**		**8420.24**

(b) MWL



Sl. No.	Particulars	Vertical Force (t)	Lever arm (m)	Moment (t-m)
1	$U_{11}=B*h_4*\gamma_w=$	681.64	B/2= 17.77	12112.52
2	$U_{12}=B*(h_2-h_4)/2*\gamma_w=$	405.50	B/3= 11.85	4803.75
	Total	**1087.14**		**16916.27**

7. Horizontal Inertia Forces/ Earthquake

Horizontal Acceleration at top of dam (c) $=1.5*\alpha_h$		
c=	0.09	
Maximum Height (h)=	47.5	m

1	For W_1		
	Horizontal Inertia Force (HIF)		
	Acceleration at bottom =	0	
	Acceleration at top (c_1) =	$c*y_1/h$	0.036
	HIF =	$W_1*c_1/3$	0.26
	L.A =	$y_1/2$	9.5
	Moment =	HIF*L.A	2.47

2	For W_2				
	Horizontal Inertia Force (HIF)				
	Acceleration at bottom =	0			
	Acceleration at top (c_2) =	$c*y_2/h$	0.050		
	HIF =	$W_2*c_2/2$	5.40		
	L.A =	$2/3y_2$	17.61		
	Moment =	HIF*L.A	95.11		

3	For W_3				
	Horizontal Inertia Force (HIF)				
	Acceleration at bottom ($c_{3\ Bottom}$) =	$c*y_2/h$	0.050		
	Acceleration at top ($c_{3\ Top}$) =	$c*Z_1/h$	0.053		
	HIF = 2/5W3*($c_{3\ Top}$ - $c_{3\ Bottom}$)	=	0.01	$W_3*c_{3\ Bottom}$	0.43
	L.A =	$y_2+4/7.y_3$	27.32	$y_2+2/5.y_3$	27.05
	Moment =	HIF*L.A	0.28		11.72

4	For W_4				
	Horizontal Inertia Force (HIF)				
	Acceleration at bottom ($c_{4\ Bottom}$) =	$c*y_5/h$	0.033		
	Acceleration at top ($c_{4\ Top}$) =	$c*Z_1/h$	0.053		
	HIF =	$W_4*(c_{4\ Top}-c_{4\ Bottom})*(m/(2m+1))$	2.215	$W_4*c_{4\ Bottom}$	9.19
	L.A =	y_5+y_4	23.373	$y_5+y_4*($	21.55

		*(2m/(3m+1))		m/(2m+1))	
	Moment =	HIF*L.A	51.783		198.13

5	**For W_5**		
	Horizontal Inertia Force (HIF)		
	Acceleration at bottom =	0	
	Acceleration at top (c_5) =	$c*y_5/h$	0.03
	HIF =	$W_5*c_5/2$	11.26
	L.A =	$(2/3)*y_5$	11.58
	Moment =	HIF*L.A	130.38

6	**For W_6**		
	Horizontal Inertia Force (HIF) =		
	Acceleration at bottom =	0	0.00
	Acceleration at top (c_6) =	$c*y_5/h$	0.03
	HIF =	$W_6*c_6/3$	3.37
	L.A =	$y_5/2$	8.68
	Moment =	HIF*L.A	29.30

7	**For W_7**				
	Horizontal Inertia Force (HIF)				
	Acceleration at bottom ($c_{7\ Bottom}$) =	$c*y_2/h$	0.050		
	Acceleration at top ($c_{7\ Top}$) =	$c*Z_1/h$	0.053		
	HIF =	$(7/10)*W_7*(c_{7\ Top}-c_{7\ Bottom})$	0.002	$W_7*c_{7\ Bottom}$	0.05
	L.A =	$y_2+38/49\ y_3$	27.643	$y_2+7/10.y_3$	27.52
	Moment =	HIF*L.A	0.063		1.49

8	**For W_8**				
	Horizontal Inertia Force (HIF)				
	Acceleration at bottom ($c_{8\ Bottom}$) =	$c*Z_1/h$	0.05		
	Acceleration at top ($c_{8\ Top}$) =	$c*(Z_1+y_6)/h$	0.08		
	HIF =	$W_8*(c_{8\ Top}-c_{8\ Bottom})/2$	1.37	$W_8*c_{8\ Bottom}$	5.48
	L.A =	$Z_1+(2/3)y_6$	37.33	$Z_1+y_6/2$	35
	Moment =	HIF*L.A	51.16		191.85

9	**For W_9**				
	Horizontal Inertia Force (HIF)				
	Acceleration at bottom ($c_{9\ Bottom}$) =	$c*(Z_1+y_6)/h$	0.080		
	Acceleration at top ($c_{9\ Top}$) =	$c*(Z_1+y_6+y_7)/h$	0.090		
	HIF =	$W_9*(c_{9\ Top}-c_{9\ Bottom})/2$	0.077	$W_9*c_{9\ Bottom}$	1.182
	L.A =	$(Z_1+y_6)+2/3.y_7$	45.667	(Z1+y6)+1/2.y7	44.750
	Moment =	HIF*L.A	3.534		52.883

10	**For W_{10}**				
	Horizontal Inertia Force (HIF)				
	Acceleration at bottom ($c_{10\ Bottom}$) =	$c*(Z_1+y_9+y_{10})/h$	0.063		
	Acceleration at top ($c_{10\ Top}$) =	$c*(Z_1+y_8+y_9+y_{10})/h$	0.080		
	HIF =	$W_{10}*(c_{10\ Top}-c_{10\ Bottom})/3$	0.075	$W_{10}*c_{10\ Bottom}$	0.890
	L.A =	$(Z_1+y_9+y_{10})+y_8/2$	37.750	$(Z_1+y_9+y_{10})+y_8/3$	36.333
	Moment =	HIF*L.A	2.842		32.345

11	**For W_{11}**				
	Horizontal Inertia Force (HIF)				
	Acceleration at bottom ($c_{11\ Bottom}$) =	$c*(Z_1+y_{10})/h$	0.059		
	Acceleration at top ($c_{11\ Top}$) =	$c*(Z_1+y_9+y_{10})/h$	0.063		
	HIF =	$W_{11}*(c_{11\ Top}-c_{11\ Bottom})/2$	0.020	$W_{11}*c_{11\ Bottom}$	0.485
	L.A =	$Z_1+y_{10}+2/3.y_9$	32.500	$Z_1+y_{10}+y_9/2$	32.250
	Moment =	HIF*L.A	0.635		15.628

12	**For W_{12}**				
	Horizontal Inertia Force (HIF)				
	Acceleration at bottom ($c_{12\ Bottom}$) =	$c*Z_1/h$	0.053		
	Acceleration at top ($c_{12\ Top}$) =	$c*(Z_1+y_{10})/h$	0.059		
	HIF =	$W_{12}*(c_{12\ Top}-c_{12\ Bottom})/2$	0.043	$W_{12}*c_{12\ Bottom}$	0.812
	L.A =	$Z_1+(2/3).y_{10}$	30.000	$Z1+y_{10}/2$	29.500
	Moment =	HIF*L.A	1.305		23.95

13	**For W_{13}**				
	Horizontal Inertia Force (HIF)				
	Acceleration at bottom ($c_{13\ Bottom}$) =	$c*y_5/h$	0.033		
	Acceleration at top ($c_{13\ Top}$) =	$c*Z_1/h$	0.053		
	HIF =	$(W_{13}*(c_{11\ Top}-c_{11\ Bottom})/2)*((3m+1)/(2m+1))$	0.490	$W_{13}*c_{11\ Bottom}$	1.15
	L.A =	$y_5+2/3.y_{11}.(11m^2+6m+1)/(3m+1)^2$	25.587	$y_5+y_{11}/2.(3m+1)/(2m+1)$	24.78
	Moment =	HIF*L.A	12.544		28.47

14	**For W_{14}**				
	Horizontal Inertia Force (HIF)				
	Acceleration at bottom ($c_{14\ Bottom}$) =	$c*y_5/h$	0.033		
	Acceleration at top ($c_{14\ Top}$) =	$c*Z_1/h$	0.053		

	HIF =	W_{14}*($c_{14\ Top}$-$c_{14\ Bottom}$)/2	0.063	W_{14}*$c_{14\ Bottom}$	0.204
	L.A =	y_5+2/3.y_{11}	24.456	y_5+y_{11}/2	22.684
	Moment =	HIF*L.A	1.529		4.632

15	**For W_{15}**				
	Horizontal Inertia Force (HIF)				
	Acceleration at bottom ($c_{15\ Bottom}$) =	c*(y_5-y_{12})/h	0.032		
	Acceleration at top ($c_{15\ Top}$) =	c*y_5/h	0.033		
	HIF =	2/3*W_{15}*($c_{15\ Top}$-$c_{15\ Bottom}$)	0.000	W_{15}*$c_{15\ Bottom}$	0.006
	L.A =	(y_5-y_{12})+3/4.y_{12}	17.217	(y_5-y_{12})+2/3.y_{12}	17.167
	Moment =	HIF*L.A	0.002		0.095

16	**Spillway Bridge**				
	HIF =	BRWT * c	1.800		
	L.A =	EL.5 - DATUM EL	44.000		
	Moment =		79.200		

Horizontal Inertia Force (HIF)

Sl. No.	Particulars	Horizontal Force (t)	Lever arm (m)	Moment (t-m)
1	On W_1	0.26	9.50	2.47
2	On W_2	5.40	17.61	95.11
3	On W_{3a}	0.01	27.32	0.28
		0.43	27.05	11.72
4	On W_4	2.22	23.37	51.78
		9.19	21.55	198.13
5	On W_5	0.03	11.58	0.38
6	On W_6	3.37	8.68	29.30
		0.00	0.00	0.00
7	On W_7	0.00	27.64	0.06
		0.05	27.52	1.49
8	On W_8	1.37	37.33	51.16
		5.48	35.00	191.85
9	On W_9	0.08	45.67	3.53
		1.18	44.75	52.88
10	On W_{10}	0.08	37.75	2.84
		0.89	36.33	32.34
11	On W_{11}	0.02	32.50	0.64
		0.48	32.25	15.63
12	On W_{12}	0.04	30.00	1.30
		0.81	29.50	23.95
13	On W_{13}	0.49	25.59	12.54
		1.15	24.78	28.47

Sl. No.	Particulars	Horizontal Force (t)	Lever arm (m)	Moment (t-m)
14	On W_{14}	0.06	24.46	1.53
		0.20	22.68	4.63
15	On W_{15}	0.00	17.22	0.00
		0.01	17.17	0.10
16	On Spillway Bridge	1.80	44.00	79.20
	Total	**35.13**		**893.33**

8. Vertical Inertia Forces/ Earthquake

Vertical Acceleration at top of Dam (c') =1.5*α_v		
c'=	0.045	
Maximum Height (h)=	47.5	m

1	**For W_1**		
	Vertical Inertia Force (VIF)		
	Acceleration at bottom =	0	
	Acceleration at top (c'_1) =	$c'*y_1/h$	0.018
	VIF =	$W_1*c'_1/3$	0.130
	L.A =	3/4.a	0.713
	Moment =	VIF*L.A	0.093

2	**For W_2**		
	Vertical Inertia Force (VIF)		
	Acceleration at bottom =	0	
	Acceleration at top (c'_2) =	$c'*y_2/h$	0.025
	VIF =	$W_2*c'_2/2$	2.701
	L.A =	a+b/2	2.653
	Moment =	VIF*L.A	7.166

3	**For W_3**				
	Vertical Inertia Force (VIF)				
	Acceleration at bottom ($c'_{3\ Bottom}$) =	$c'*y_2/h$	0.025		
	Acceleration at top (c'_{Top}) =	$c'*Z_1/h$	0.027		
	VIF =	$2/5W_3*(c'_{3\ Top}-c'_{3\ Bottom})$	0.005	$W_3*c'_{3\ Bottom}$	0.22
	L.A =	a+b-5/16.b	3.292	a+b-3/8.b	3.08
	Moment =	VIF*L.A	0.017		0.67

4	**For W_4**				
	Vertical Inertia Force (VIF)				
	Acceleration at bottom ($c'_{4\ Bottom}$) =	$c'*y_5/h$	0.016		
	Acceleration at top ($c'_{4\ Top}$) =	$c'*Z_1/h$	0.027		
	VIF =	$W_4*(c'_{4\ Top}-c'_{4}$	1.108	$W_4*c'_{4\ Bottom}$	4.596

		Bot-tom).m/(2m+1)			
	L.A =	a+b+(c/4)*(2m+1)/(m+2)	9.368	a+b+(c/2)*(m+1)/(m+2)	10.434
	Moment =	VIF*L.A	10.377		47.96

5	For W_5		
	Vertical Inertia Force (VIF)		
	Acceleration at bottom =	0	
	Acceleration at top (c'_5) =	$c'*y_5/h$	0.016
	VIF =	$W_5*c'_5/2$	5.631
	L.A =	a+b+c/2	12.567
	Moment =	VIF*L.A	70.758

6	For W_6		
	Vertical Inertia Force (VIF)		
	Acceleration at bottom ($c'_{6\ Bottom}$) =	0	0.000
	Acceleration at top ($c'_{6\ Top}$) =	$c'*y_5/h$	0.016
	VIF =	$W_6*c'_6/3$	1.687
	L.A =	a+b+c+d/4	24.468
	Moment =	VIF*L.A	41.283

7	For W_7				
	Vertical Inertia Force (VIF)				
	Acceleration at bottom ($c'_{7\ Bottom}$) =	$c'*y_2/h$	0.025		
	Acceleration at top ($c'_{7\ Top}$) =	$c'*Z_1/h$	0.027		
	VIF =	$(7/10)*W_7*(c'_{7\ Top}-c'_{7\ Bottom})$	0.001	$W_7*c'_{7\ Bottom}$	0.027
	L.A =	$a+b-5b/_7$	1.923	a+b/4	1.802
	Moment =	VIF*L.A	0.002		

8	For W_8				
	Vertical Inertia Force (VIF)				
	Acceleration at bottom ($c'_{8\ Bottom}$) =	$c'*Z_1/h$	0.027		
	Acceleration at top ($c'_{8\ Top}$) =	$c'*(Z_1+y_6)/h$	0.040		
	VIF =	$W_8*(c'_{8\ Top}-c'_{8\ Bottom})/2$	0.685	$W_8*c'_{8\ Bottom}$	2.741
	L.A =	$a+(x_2+x_3)/2$	7.100	$a+(x_2+x_3)/2$	7.100
	Moment =	VIF*L.A	4.865		19.459

9	For W_9				
	Vertical Inertia Force (VIF)				
	Acceleration at bottom ($c'_{9\ Bottom}$) =	$c'*(Z_1+y_6)/h$	0.040		

	Acceleration at top ($c'_{9\ Top}$) =	$c'*(Z_1+y_6+y_7)/h$	0.045		
	VIF =	$W_9*(c'_{9\ Top}-c'_{9\ Bottom})/2$	0.039	$W_9*c'_{9\ Bottom}$	0.591
	L.A =	$a+x_2+x_3/2$	11.000	$a+x_2+x_3/2$	11.000
	Moment =	VIF*L.A	0.426		6.500

10	**For W_{10}**				
	Vertical Inertia Force (VIF)				
	Acceleration at bottom ($c'_{10\ Bottom}$) =	$c'*(Z_1+y_9+y_{10})/h$	0.032		
	Acceleration at top ($c'_{10\ Top}$) =	$c'*(Z_1+y_8+y_9+y_{10})/h$	0.040		
	VIF =	$W_{10}*(c'_{10\ Top}-c'_{10\ Bottom})/3$	0.038	$W_{10}*c'_{10\ Bottom}$	0.445
	L.A =	$a+x_2+x_3+x_4/4$	14.625	$a+x_2+x_3+x_4/3$	15.083
	Moment =	VIF*L.A	0.551		6.714

11	**For W_{11}**				
	Vertical Inertia Force (VIF)				
	Acceleration at bottom ($c'_{11\ Bottom}$) =	$c'*(Z_1+y_{10})/h$	0.029		
	Acceleration at top ($c'_{11\ Top}$) =	$c'*(Z_1+y_9+y_{10})/h$	0.032		
	VIF =	$W_{11}*(c'_{11\ Top}-c'_{11\ Bottom})/2$	0.010	$W_{11}*c'_{11\ Bottom}$	0.242
	L.A =	$a+x_2+x_3+x_4/2$	16.000	$a+x_2+x_3+x_4/2$	16.000
	Moment =	VIF*L.A	0.156		3.877

12	**For W_{12}**				
	Vertical Inertia Force (VIF)				
	Acceleration at bottom ($c'_{12\ Bottom}$) =	$c'*Z_1/h$	0.027		
	Acceleration at top ($c'_{12\ Top}$) =	$c'*(Z_1+y_{10})/h$	0.029		
	VIF =	$W_{12}*(c'_{12\ Top}-c'_{12\ Bottom})/2$	0.022	$W_{12}*c'_{12\ Bottom}$	0.406
	L.A =	$a+x_2+x_3+(x_4+x_5)/2$	17.500	$a+x_2+x_3+(x_4+x_5)/2$	17.500
	Moment =	VIF*L.A	0.380		7.102

13	**For W_{13}**				
	Vertical Inertia Force (VIF)				
	Acceleration at bottom ($c'_{13\ Bottom}$) =	$c'*y_5/h$	0.016		
	Acceleration at top ($c'_{13\ Top}$) =	$c'*Z_1/h$	0.027		
	VIF =	$W_{13}*((c'_{11\ Top}-c'_{11\ Bottom})/2)*(3m+1)/(2m+1)$	0.245	$W_{13}*c'_{13\ Bottom}$	0.575
	L.A =	a+b+(c/2)*(3m	15.910	a+b+c.(m+	16.512

		+2)(2m+1)/(3m +1)(m+2)		1)/(m+2)	
	Moment =	VIF*L.A	3.900		9.487

14	**For W_{14}**				
	Vertical Inertia Force (VIF)				
	Acceleration at bottom ($c'_{14\ Bottom}$) =	$c'*y_5/h$	0.016		
	Acceleration at top ($c'_{14\ Top}$) =	$c'*Z_1/h$	0.027		
	VIF =	$W_{14}*(c'_{14\ Top}-c'_{14\ Bottom})/2$	0.031	$W_{14}*c'_{14\ Bottom}$	0.10
	L.A =	a+b+c+aa/2	21.264	a+b+c+aa/2	21.26
	Moment =	VIF*L.A	0.665		2.17

15	**For W_{15}**				
	Vertical Inertia Force (VIF)				
	Acceleration at bottom (c'15 Bottom) =	$c'*(y_5-y_{12})/h$	0.016		
	Acceleration at top (c'15 Top) =	$c'*y_5/h$	0.016		
	VIF =	$(2/3)*W_{15}*(c'_{15\ Top}-c'_{15\ Bottom})$	0.000	$W_{15}*c'_{15\ Bottom}$	0.00
	L.A =	a+b+c+5aa/8	21.385	a+b+c+2aa/3	21.43
	Moment =	VIF*L.A	0.001		0.06

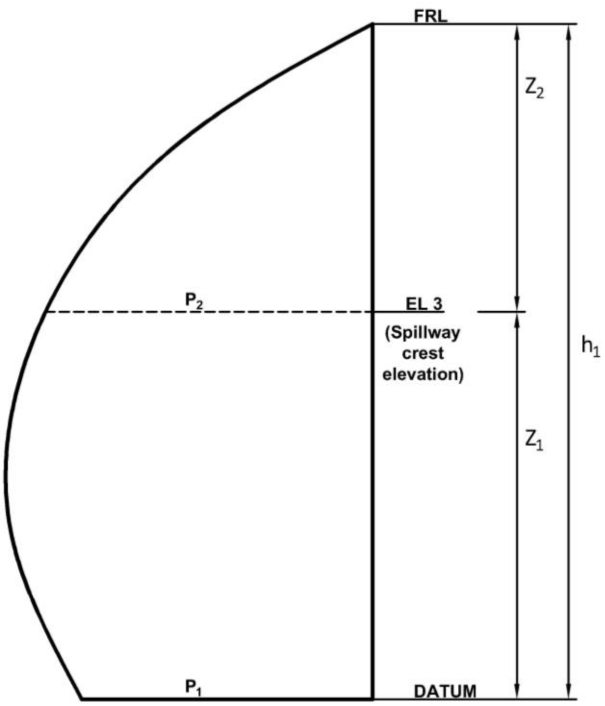
16	**Spillway Bridge**		
	VIF=	BRWT*c'	0.900
	L.A =	a+CW+TW/2	5.250
	Moment =	VIF*L.A	4.725

Vertical Inertia Force (VIF)

Sl. No.	Particulars	Vertical Force (t)	Lever arm (m)	Moment (t-m)
1	On W_1	0.13	0.71	0.09
2	On W_2	2.70	2.65	7.17
3	On W_3	0.01	3.29	0.02
		0.22	3.08	0.67
4	On W_4	1.11	9.37	10.38
		4.60	10.43	47.96
5	On W_5	0.02	12.57	0.21
6	On W_6	1.69	24.47	41.28
7	On W_7	0.00	1.92	0.00
		0.03	1.80	0.05
8	On W_8	0.69	7.10	4.86
		2.74	7.10	19.46
9	On W_9	0.04	11.00	0.43
		0.59	11.00	6.50
10	On W_{10}	0.04	14.63	0.55

Sl. No.	Particulars	Vertical Force (t)	Lever arm (m)	Moment (t-m)
		0.45	15.08	6.71
11	On W_{11}	0.01	16.00	0.16
		0.24	16.00	3.88
12	On W_{12}	0.02	17.50	0.38
		0.41	17.50	7.10
13	On W_{13}	0.25	15.91	3.90
		0.57	16.51	9.49
14	On W_{14}	0.03	21.26	0.66
		0.10	21.26	2.17
15	On W_{15}	0.00	21.39	0.00
		0.00	21.43	0.06
16	On Spillway Bridge	0.90	5.25	4.73
	Total	**17.56**		**178.86**

9. Hydrodynamic Pressure Due to Earthquake



C_m =	0.735		
At any depth y, Cs =	$C_m/2[y/h(2-y/h)+sqrt\{y/h(2-y/h)\}]$		
At foundation level,			
h_1 =	40.0	m	
$y = h_1$=	40.0	m	
$h = h_1$=	40.0	m	
C_{s1} =	$C_m/2[y/h(2-y/h)+sqrt\{y/h(2-y/h)\}]$ = 0.735		
Hydrodynamic Pressure (p_1) =	$C_{s1}*\alpha_h*\gamma_w*h$ =	1.76	t/m^2
Hydrodynamic force=V_1 =	$0.726*p_1*y$ =	51.23	t

At EL.3,			
$y = z_2 =$	12.0	m	
$h = h_1 =$	40.0	m	
$C_{s2} =$	$C_m/2[y/h(2-y/h)+sqrt\{y/h(2-y/h)\}] = 0.450$		
Hydrodynamic Pressure (p_2) =	$C_{s2}*\alpha_h*\gamma_w*h$ =	1.08	t/m^2
Hydrodynamic force=V_2 =	$0.726*p_2*y$ =	9.41	t

Calculation of Hydrodynamic Shear/ Moments at Foundation Level

(a)	**Pier portion of the block**		
	$V_h = 0.726*p_1*h_1*PW/BW =$	12.807	t
	$M_h = 0.299*p_1*h_1^2*PW/BW =$	210.974	t-m

(b)	**Non-Pier portion of the block**		
	(i) Area between FRL and EL.3 (Gates portion)		
	$V_h = V_2*(BW-PW)/BW =$	7.055	t
	$M_h = V_h*(EL\ 12 - DATUM\ EL.) =$	225.751	t-m
	(ii) Area between EL.3 and DATUM EL.		
	$V_h = (V_1-V_2)*(BW-PW)/BW =$	23.524	t
	$M_h = [0.299*p_1*h_1^2-V_2\{Z_1+0.299*z_2/0.726\}]*(BW-PW)/BW =$	400.526	t-m

Load Combinations

1. Load Combination A: (Reservoir Empty)

Sl. No.	Particulars	Vertical Force (t)	Lever arm (m)	Moment (t-m)
1	Self-Weight of the dam	1735.81		22153.23
	Total	**1735.81**	**12.76**	**22153.23**

$x = (\Sigma M/\Sigma W) =$	12.76	m	
$e = (B/2-x) =$	5.01	m	
$6e/B =$	0.85		
Stress at u/s= $\Sigma W(1+6e/B)/B$		90.131	t/m^2
Stress at d/s= $\Sigma W(1-6e/B)/B$		7.554	t/m^2

2. Load Combination B: (Reservoir at FRL, Normal Uplift, without Earthquake, Min TWL)

Sl. No.	Particulars	Vertical Force (t)	Horizontal Force (t)	Lever arm (m)	Moment (t-m)
1	Self-Weight of the dam	1735.81			22153.23
2	Horizontal water pressure		800.00		10666.67
3	Weight of water	68.63			137.36
4	Horizontal silt pressure		89.11		660.91
5	Weight of Silt	11.20			0.00
6	Tail Water Pressure		0		0.00
7	Weight of tail water	0.00			0.00

Sl. No.	Particulars	Vertical Force (t)	Horizontal Force (t)	Lever arm (m)	Moment (t-m)
8	Uplift pressure (Normal)	-326.93			-3297.14
	Total	**1488.72**	**889.11125**	**20.37**	**30321.03**

x = (ΣM/ΣW) =	20.37	m	
e= (B/2-x) =	-2.60	m	
6e/B =	-0.44		
Stress at u/s= ΣW(1+6e/B)/B		23.52	t/m^2
Stress at d/s= ΣW(1-6e/B)/B		60.26	t/m^2

$F = \left(\sum \dfrac{(w-u)\tan\phi}{F\phi} + \dfrac{CA}{Fc} \right) \div H$	=	1.34

F= factor of safety against sliding	
tan ø = Coefficient of internal friction of the material	0.7000
C= Cohesion at contact between rock and concrete	50
A= area under consideration for cohesion	35.54
Fø= partial factor of safety in respect of friction	1.5
Fc= partial factor of safety in respect of cohesion	3.6

3. Load Combination C: (Reservoir at MWL, Normal Uplift, without Eathquake, Max. TWL)

Sl. No.	Particulars	Vertical Force (t)	Horizontal Force (t)	Lever Arm (m)	Moment (t-m)
1	Self Weight of the dam	1735.81			22153.23
2	Horizontal water pressure		808.50		9947.00
3	Weight of water	30.88			13.24
4	Horizontal silt pressure		89.11		660.91
5	Weight of Silt	11.20			0.00
6	Tail Water Pressure		-183.94		-1175.97
7	Weight of tail water	0.00			0.00
8	Uplift pressure (Normal)	-868.15			-13993.54
	Total	**909.74**	**713.68**	**19.35**	**17604.87**

x = (ΣM/ΣW) =	19.35	m	
e= (B/2-x) =	-1.58	m	
6e/B =	-0.27		
Stress at u/s= ΣW(1+6e/B)/B		18.76	t/m^2
Stress at d/s= ΣW(1-6e/B)/B		32.43	t/m^2

$F = \left(\sum \dfrac{(w-u)\tan\phi}{F\phi} + \dfrac{CA}{Fc} \right) \div H$	=	**1.29**

F= factor of safety against sliding	
tan ø = Coefficient of internal friction of the material	0.7000
C= Cohesion at contact between rock and concrete	50
A= area under consideration for cohesion	35.54
Fø= partial factor of safety in respect of friction	1.5
Fc= partial factor of safety in respect of cohesion	3.6

4. Load combination D: (Combination A, with Earthquake)

Sl. No.	Particulars	Vertical Force (t)	Horizontal Force (t)	Lever arm (m)	Moment (t-m)
1	Load combination A	1735.81			22153.23
2	Horizontal Inertia Force (HIF)		-35.13		-893.33
3	Vertical Inertia Force (VIF)	17.56			178.86
	Total	**1753.37**	**-35.13**	**12.23**	**21438.75**

x = (ΣM/ΣW) =	12.23	m		
e= (B/2-x) =	5.54	m		
6e/B =	0.94			
Stress at u/s= ΣW(1+6e/B)/B		95.50	t/m^2	
Stress at d/s= ΣW(1-6e/B)/B		3.17	t/m^2	

$$F = \left(\sum \frac{(w-u)\tan\phi}{F\phi} + \frac{CA}{Fc} \right) \div H \quad = \quad 50.20$$

F= factor of safety against sliding	
tan ø = Coefficient of internal friction of the material	0.7000
C= Cohesion at contact between rock and concrete	50
A= area under consideration for cohesion	35.54
Fø= partial factor of safety in respect of friction	1.2
Fc= partial factor of safety in respect of cohesion	2.4

5. Load Combination E: (Combination B, with Earthquake)

Sl. No.	Particulars	Vertical Force (t)	Horizontal Force (t)	Lever arm (m)	Moment (t-m)
1	Load combination B	1488.72	889.11		30321.03
2	Horizontal Inertia Force (HIF)		35.13		893.33
3	Vertical Inertia Force (VIF)	-17.56			-178.86
4	Hydrodynamic Pressure		43.39		837.25
	Total	**1471.15**	**924.24**	**21.10**	**31035.50**

x = (ΣM/ΣW) =	21.10	m	
e= (B/2-x) =	-3.33	m	
6e/B=	-0.56		

Stress at u/s= ΣW(1+6e/B)/B		18.15	t/m^2
Stress at d/s= ΣW(1-6e/B)/B		64.64	t/m^2

$F = \left(\sum \frac{(w-u)\tan ø}{Fø} + \frac{CA}{Fc} \right) \div H$	=	1.73

F= factor of safety against sliding	
tan ø = Coefficient of internal friction of the material	0.7000
C= Cohesion at contact between rock and concrete	50
A= area under consideration for cohesion	35.54
Fø= partial factor of safety in respect of friction	1.2
Fc= partial factor of safety in respect of cohesion	2.4

6. **Load Combination F: (Combination C, but Drains Choked)**

Sl. No	Particulars	Vertical Force (t)	Horizontal Force (t)	Lever arm (m)	Moment (t-m)
1	Load Combination C	909.736	713.675		17608.87
2	Less Normal Uplift	868.155			13993.54
3	Add Extreme Uplift	-1087.145			-16916.27
	Total	**690.746**	**713.675**	**21.261**	**14686**

x = (ΣM/ΣW) =	21.26	m	
e= (B/2-x) =	-3.49	m	
6e/B =	-0.59		
Stress at u/s= ΣW(1+6e/B)/B		7.98	t/m^2
Stress at d/s= ΣW(1-6e/B)/B		30.89	t/m^2

$F = \left(\sum \frac{(w-u)\tan ø}{Fø} + \frac{CA}{Fc} \right) \div H$	=	2.75

F= factor of safety against sliding	
tan ø = Coefficient of internal friction of the material	0.7000
C= Cohesion at contact between rock and concrete	50
A= area under consideration for cohesion	35.54
Fø= partial factor of safety in respect of friction	1
Fc= partial factor of safety in respect of cohesion	1.2

7. **Load Combination G: (Combination E, but Drains choked)**

Sl. No.	Particulars	Vertical Force (t)	Horizontal Force (t)	Lever arm (m)	Moment (t-m)
1	Load Combination E	1471.154	924.236		31039.50
2	Less Normal Uplift	326.928			3297.14
3	Add Extreme Uplift	-710.785			-8420.24

	Total	**1087.298**	**924.236**	**23.836**	**25916.40**

x = (ΣM/ΣW) =	23.84	m
e= (B/2-x) =	-6.07	m
6e/B=	-1.02	

Stress at u/s= ΣW(1+6e/B)/B	-0.74	t/m^2
Stress at d/s= ΣW(1-6e/B)/B	61.93	t/m^2

$F = \left(\sum \frac{(w-u)\tan ø}{Fø} + \frac{CA}{Fc} \right) \div H$	=	2.43

F= factor of safety against sliding	
tan ø = Coefficient of internal friction of the material	0.7000
C= Cohesion at contact between rock and concrete	50
A= area under consideration for cohesion	35.54
Fø= partial factor of safety in respect of friction	1.0
Fc= partial factor of safety in respect of cohesion	1.2

Summary of Results for OF Section at EL.541

Load Combination	Vertical Stress at Upstream (t/m^2)	Vertical Stress at Downstream (t/m^2)	Factor of safety Against Sliding	Allowable Tensile Stress (t/m^2)
A	90.13	7.55		-
B	23.50	60.28	1.34	No Tension
C	18.74	32.45	1.29	15
D	95.50	3.17	50.20	
E	18.13	64.66	1.73	30
F	7.98	30.89	2.75	30
G	-0.74	61.93	2.43	60

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APPENDIX B - AID TO CALCULATION OF INERTIA FORCES AND MOMENTS DUE TO EARTHQUAKE

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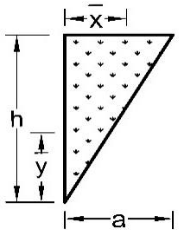
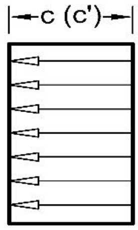
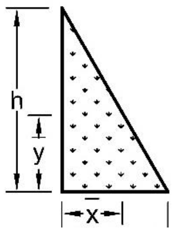
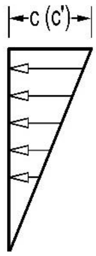
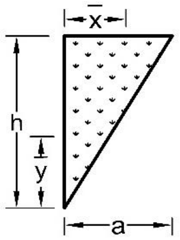
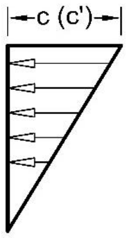
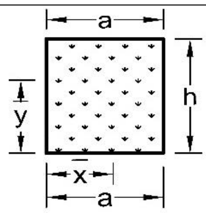
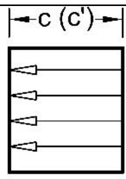
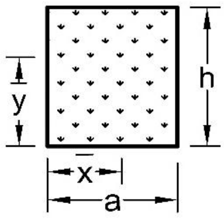
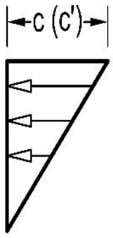
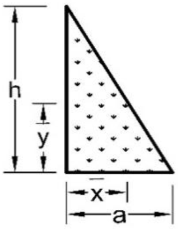
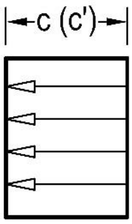
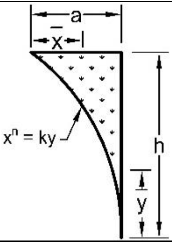
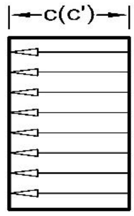
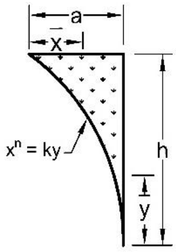
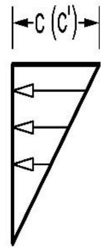
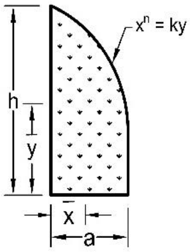
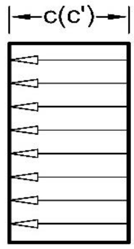
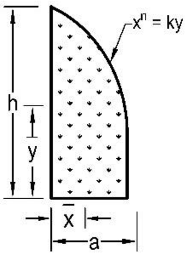
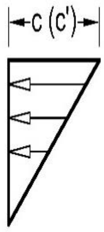
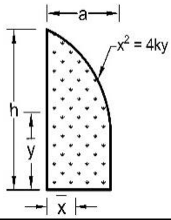
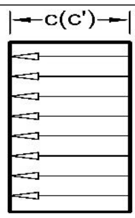
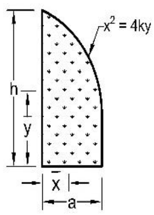
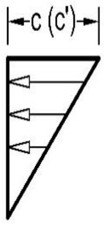
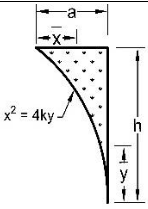
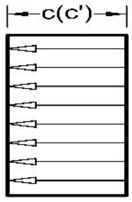
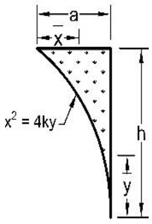
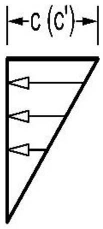
AID TO CALCULATION OF INERTIA FORCES AND MOMENTS DUE TO EARTH-QUAKE

S.no	Horizontal	Mass	Acceleration Distribution	Vertical
1	$F=Wc$ $\bar{y}=\frac{2h}{3}$ $M=\frac{2Wch}{3}$			$F=W\acute{c}$ $\bar{x}=\frac{a}{3}$ $M=\frac{W\acute{c}a}{3}$
2	$F=\frac{Wc}{3}$ $\bar{y}=\frac{h}{2}$ $M=\frac{Wch}{2}$			$F=\frac{W\acute{c}}{3}$ $\bar{x}=\frac{a}{4}$ $M=\frac{W\acute{c}a}{12}$
3	$F=\frac{2Wc}{3}$ $\bar{y}=\frac{3h}{4}$ $M=\frac{Wch}{2}$			$F=\frac{2W\acute{c}}{3}$ $\bar{x}=\frac{3a}{8}$ $M=\frac{W\acute{c}a}{4}$
4	$F=Wc$ $\bar{y}=\frac{h}{2}$ $M=\frac{Wch}{2}$			$F=W\acute{c}$ $\bar{x}=\frac{a}{2}$ $M=\frac{W\acute{c}a}{2}$
5	$F=\frac{Wc}{2}$ $\bar{y}=\frac{2h}{3}$ $M=\frac{Wch}{3}$			$F=\frac{W\acute{c}}{2}$ $\bar{x}=\frac{a}{2}$ $M=\frac{W\acute{c}a}{2}$
6	$F=Wc$ $\bar{y}=\frac{h}{3}$ $M=\frac{Wch}{3}$			$F=W\acute{c}$ $\bar{x}=\frac{a}{3}$ $M=\frac{W\acute{c}a}{3}$

7	F=Wc $\bar{y} = \frac{h}{2}\left(\frac{3n+1}{2n+1}\right)$ [0.6968085h for n=1.85] $M = \frac{Wch}{2}\left(\frac{3n+1}{2n+1}\right)$			F= Wć $\bar{x} = a\left(\frac{n+1}{n+2}\right)$ [0.7402597a for n=1.85] $M = W\acute{c}a\left(\frac{n+1}{n+2}\right)$
8	$F = \frac{Wc}{2}\left(\frac{3n+1}{2n+1}\right)$ [0.6968085h for n=1.85] $\bar{y} = \frac{2h}{3}\left(\frac{11n^2+6n+1}{(3n+1)^2}\right)$ [0.7730318h for n=1.85] $M = \frac{Wch}{3}\left(\frac{11n^2+6n+1}{(2n+1)(3n+1)}\right)$			$F = \frac{W\acute{c}}{2}\left(\frac{3n+1}{2n+1}\right)$ [0.6968085Wć for n=1.85] $\bar{x} = \frac{a}{2}\left(\frac{3n+2}{3n+1}\right)\left(\frac{2n+1}{n+2}\right)$ [0.7035788a for n=1.85] $M = \frac{W\acute{c}a}{4}\left(\frac{3n+2}{n+2}\right)$
9	$F = Wc\left(= \frac{nah}{n+1}c\right)$ $\bar{y} = \frac{nh}{2n+1}$ [0.393617h for n=1.85] $M = Wc\left(\frac{nh}{2n+1}\right)$			F= Wć $\bar{x} = \frac{a}{2}\left(\frac{n+1}{n+2}\right)$ [0.3701298a for n=1.85] $M = \frac{W\acute{c}a}{2}\left(\frac{n+1}{n+2}\right)$
10	$F = Wc\left(= \frac{n}{2n+1}\right)$ [0.393617h for n=1.85] $\bar{y} = \frac{2nh}{3n+1}$ [0.5648854h for n=1.85] $M = Wc\left(\frac{2n^2}{(2n+1)(3n+1)}\right)$			$F = W\acute{c}\left(\frac{n}{2n+1}\right)$ [0.393617h for n=1.85] $\bar{x} = \frac{a}{4}\left(\frac{2n+1}{n+2}\right)$ [0.3051947a for n=1.85] $M = \frac{Wc\,\acute{a}n}{4(n+2)}$
11	$F = Wc$ $\bar{y} = \frac{2h}{5}$ $M = \frac{2Wch}{5}$			F= Wć $\bar{x} = \frac{3a}{8}$ $M = \frac{3W\acute{c}a}{8}$
12	$F = \frac{2Wc}{5}$ $\bar{y} = \frac{4h}{7}$ $M = \frac{8Wch}{35}$			$F = \frac{2Wc'}{5}$ $\bar{x} = \frac{5a}{16}$ $M = \frac{W\acute{c}a}{8}$

13	$\mathbf{F = Wc}$ $\bar{y} = \frac{7h}{10}$ $\mathbf{M} = \frac{7Wch}{10}$			$\mathbf{F = Wc'}$ $\bar{x} = \frac{3a}{4}$ $\mathbf{M} = \frac{3Wc'a}{4}$
14	$\mathbf{F} = \frac{7Wc}{10}$ $\bar{y} = \frac{38h}{49}$ $\mathbf{M} = \frac{19Wch}{35}$			$\mathbf{F} = \frac{7Wc'}{10}$ $\bar{x} = \frac{5a}{7}$ $\mathbf{M} = \frac{Wc'a}{2}$

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APPENDIX C – DESIGN EARTHQUAKES AND GROUND MOTIONS

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DESIGN EARTHQUAKES AND GROUND MOTIONS

It is common to consider two levels of ground motion (GM) with corresponding performance requirements in the seismic design of a new dam. However, the terminology for these two GM levels and their definition has not been standardized. We first summarize the essentially identical terminology and definitions in publications of two major organizations: International Commission on Large Dams (ICOLD) and Federal Emergency Managements Agency (FEMA) in the United States [ICOLD, 2016; FEMA 2014].

C.1 ICOLD and FEMA

The Operating Basis Earthquake (OBE) is the earthquake event that produces GM at the site that can reasonably be expected during the service life of the project. This statement has usually been interpreted as GM that has a 50% probability of exceedance (PE) in 100 years, the commonly assumed life of concrete dams. The corresponding mean return period is 144 years (calculated assuming a Poisson model for occurrence of events). At this level of ground shaking, the facility—dam, appurtenant structures, equipment, power house, etc.—should experience little or no damage and continue to function without interruption; this performance requirement implies that the dam remains essentially within the linear range of behavior. The OBE should be determined by Probabilistic Seismic Hazard Analysis (PSHA).

The Safety Evaluation Earthquake (SEE) or Maximum Design Earthquake (MDE) is the earthquake event that produces GM at the site that is rare. Factors to consider in selecting the intensity of this GM are the consequences of failure of the dam, criticality of project function (power generation, water supply, flood control, etc.), and turnaround time to restore the facility to be operational after the earthquake event. The MDE represents ground shaking at the site associated with a long mean return period: 10,000, 3000, or 1000 years for dams where the consequences of dam failure are high, moderate, or low, respectively. Mean return periods of 10,000 (precisely 9950) years and 1000 (precisely 949 years) represent ground shaking associated with a 1% and 10% PE in 100 years, respectively. The MDE should also be determined by PSHA. At this level of ground shaking, there should be no catastrophic failure, such as uncontrolled release of the impounded water, although significant damage or economic loss may be tolerated. This performance requirement implies that the dam is allowed to deform significantly into the nonlinear range.

The FEMA and ICOLD documents also define a Maximum Credible Earthquake (MCE). This represents the GM during the largest magnitude earthquake along a recognized fault or within a particular seismo-tectonic province. A deterministic approach is used to determine the MCE ground motions at the site. For each identified fault, the largest magnitude earthquake is used as input to a ground motion prediction model (GMPM) to provide the probability distribution of the GM. The 84^{th} percentile value is defined as the deterministic-based MCE-level motion.

At sites close to major faults with high-slip rates (e.g., the San Andreas and Hayward faults in California), the earthquake event that produces the MCE-level GM may have a relatively high annual rate of occurrence, e.g., 0.015[+] for the Hayward fault in California. Combining the annual rate of 0.015 with a GM with 16% probability of being exceeded results in a return period of

[+]The Hayward fault has a 31.7% chance of rupturing in a 6.7 or larger magnitude earthquake in the next 26 years; http://seismo.berkeley.edu/hayward/hayward_hazards.html. The associated annual rate of occurrence is 0.015.

416++ years, a much, much shorter return period than the 10,000 years for the MDE. This implies that when the next large earthquake occurs on a major fault in California, the MCE-level GM would, on average, be exceeded at 16% of the dams. This does not seem to be prudent, suggesting that the MCE-level (84th percentile deterministic) GM is not strong enough. However, in other parts of the world where the slip rates on active faults are low, the MCE-level GM may be much more intense than the GM with a 10,000-year return period. Therefore, for safety evaluation of high-consequence dams the more intense of two GMs should be selected: (1) GM from MCE on known active faults; and (2) GM associated with 1% PE in 100 years or return period of 10,000 years [ANCOLD, 2017].

C.2 U.S. Army Corps of Engineers [USACE, 1999; 2016]

The definition for the OBE is identical to the one stated in the last section, but the MDE is defined differently. For critical features of the project, the MDE is the same as the MCE. For all other features, the minimum MDE is an event with a 10% probability of exceedance in 100 years, implying a mean return period of 950 (precisely 949) years. A shorter or longer return period for non-critical features may be appropriate, depending on the consequences associated with failure or the dam.

C.3 Division of Safety of Dams (DSOD), State of California

This influential agency continues to use deterministic methods to define seismic hazard. The MCE-level ground motion is defined as the 84th percentile estimate from the ground-motion prediction models (GMPMs)

C.4 U.S. Federal Energy Regulatory Commission (FERC)

Using deterministic methods to define seismic hazard, FERC requires dynamic analysis of the dam for the MCE ground motion followed by static analysis to evaluate the post-seismic stability of the damaged dam with reduced shear strengths and increased uplift pressures, resulting from damage to drains. FERC is concerned only about an uncontrolled release of the impounded water but not with operability of the facility at ground motions of lower intensity.

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U.S. Army Corps of Engineers (1999) Response spectra and seismic analysis for concrete hydraulic structures, *Design Manual EM1110-2-6050*, Washington, D.C.

U.S. Army Corps of Engineers (2016). Earthquake design and evaluation for civil works projects, *Design Manual EMR1110-2-1806*, Washington, D.C.

++ **Return period = $(0.015 \times 0.16)^{-1}$ = 416 years.**

APPENDIX D –RESPONSE SPECTRUM ANALYSIS OF CONCRETE GRAVITY DAMS INCLUDING DAM-WATER-FOUNDATION INTERACTION

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PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

Response Spectrum Analysis of Concrete Gravity Dams Including Dam-Water-Foundation Interaction

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PEER 2013/17
JULY 2013

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Headquarters at the University of California, Berkeley

July 2013

ABSTRACT

A response spectrum analysis (RSA) procedure, which estimates the peak response directly from the earthquake design spectrum, was developed in 1986 for the preliminary phase of design and safety evaluation of concrete gravity dams. The analysis procedure includes the effects of dam-water-foundation interaction, known to be important in the earthquake response of dams.

This report presents a comprehensive evaluation of the accuracy of the RSA procedure by comparing its results with those obtained from response history analysis (RHA) of the dam modeled as a finite element system, including dam-water-foundation interaction. The earthquake response of an actual dam to an ensemble of 58 ground motions, selected and scaled to be consistent with a target spectrum determined from a probabilistic seismic hazard analysis for the dam site, was determined by the RHA procedure. The median of the peak responses of the dam to 58 ground motions provided the benchmark result. The peak response was also estimated by the RSA procedure directly from the median response spectrum. Comparison of the two sets of results demonstrated that the RSA procedure estimates stresses to a degree of accuracy that is satisfactory for the preliminary phase in the design of new dams and in the safety evaluation of existing dams. The accuracy achieved in the RSA procedure is noteworthy, especially considering the complicated effects of dam-water-foundation interaction and reservoir bottom absorption on the dynamics of the system, and the number of approximations necessary to develop the procedure.

Also developed in the report is a more complete set of data for the parameters that characterize dam-foundation interaction in the RSA procedure. Availability of these data should provide sufficient control over the overall damping in the dam-water-foundation system to ensure consistency with damping measured from motions of dams recorded during forced vibration tests and earthquakes.

ACKNOWLEDGMENTS

We are grateful to several individuals who helped in this research:

- Professor Gautam Dasgupta at Columbia University provided the computer program to compute new compliance data for a viscoelastic half-plane.
- Professor Baris Binici at the Middle East Technical University (METU) provided a set of Matlab scripts that were used as the starting point to develop pre- and post-processors to the EAGD-84 computer program, which was utilized to perform all the response history analyses presented in this report.
- Professor Pierre Léger at École Polytechnique de Montréal incorporated the new data presented in this report into the "pseudo-dynamic procedure" in the widely used computer program CADAM.

The first author would like to thank the Norwegian National Committee on Large Dams (NNCOLD) and Norwegian Water for their financial support during his visit to the University of California, Berkeley, in 2013 when this report was prepared.

The publication of this report by the Pacific Earthquake Engineering Research Center (PEER) is gratefully acknowledged. Any opinions, findings, and conclusions or recommendations expressed in this material are those of the authors and do not necessarily reflect those of PEER or any other sponsors.

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1 Introduction

The elastic analysis phase of seismic design and safety evaluation of concrete gravity dams may be organized in two stages [Chopra 1978]: (1) response spectrum analysis (RSA) in which the peak value, i.e., the maximum absolute value, of response is estimated directly from the earthquake design spectrum; and (2) response history analysis (RHA) of a finite element idealization of the dam monolith. The RSA procedure was recommended for the preliminary phase of design and safety evaluation of dams, and the RHA procedure for accurately computing the dynamic response and checking the adequacy of the preliminary evaluation. Dam-water interaction effects were included in both procedures [Chopra 1978, Chakrabarti and Chopra 1973].

In the mid 1980s, both procedures were extended to consider absorption of hydrodynamic pressure waves into the alluvium and sediments invariably deposited at the bottom of reservoirs and, more importantly, interaction between the dam and underlying foundation [Fenves and Chopra 1984b, 1987]. Recognizing that the cross-sectional geometry of concrete gravity dams does not vary widely, standard data for the vibration properties of dams and parameters characterizing dam-water-foundation interaction effects were presented to facilitate the implementation of the RSA procedure [Fenves and Chopra 1987]. Both the RSA procedure, implemented in CADAM [Leclerc, Legér, and Tinawi 2003], and the RHA procedure, implemented in the computer program EAGD–84 [Fenves and Chopra 1984c], have been utilized extensively in seismic design of new dams and seismic evaluation of existing dams.

This report presents a comprehensive evaluation of the accuracy of the RSA procedure, in contrast to the limited scope of the earlier investigation [Fenves and Chopra 1987]. To enhance the accuracy of this RSA procedure, the possibility of calculating stresses by finite element analysis versus the commonly used beam formulas is explored, and a correction factor for beam stresses on the downstream face of the dam is developed. Also included is a more complete set of data for the parameters that characterize dam-foundation interaction. This was motivated by the realization that viscous damping of 5%, commonly assumed for rock, may be excessive, and that data presented earlier did not provide sufficient control over the overall damping in the dam-water-foundation system to ensure consistency with damping measured from motions of dams recorded during forced vibration tests and earthquakes [Rea, Liaw, and Chopra 1975; Proulx et. al. 2001; Alves and Hall 2006]. For the sake of completeness, the RSA procedure is summarized and standard values for parameters that characterize dam-water interaction and reservoir bottom absorption are included, thus making this report self-contained.

2 Response Spectrum Analysis Procedure

The response spectrum analysis (RSA) procedure developed to estimate the earthquake-induced stresses in concrete gravity dams considers only the more significant aspects of the response. Although the dynamics of the system including dam-water-foundation interaction is considered in estimating the response due to the fundamental vibration mode, the less significant part of the response due to higher modes is estimated by the static correction method. Only the horizontal component of ground motion is considered because the response due to the vertical component is known to be much smaller [Fenves and Chopra 1984a].

Dam-water-foundation interaction introduces frequency-dependent, complex-valued hydrodynamic and foundation terms in the governing equations. Based on a clever series of approximations, frequency-independent values of these terms were defined and an equivalent SDF system developed to estimate the fundamental mode response of dams, leading to the RSA procedure summarized in the subsequent sections. This development was presented and approximations evaluated and justified in a series of publications [Fenves and Chopra 1985a, 1985b, 1987].

The two-dimensional system considered consists of a concrete gravity dam monolith supported on a horizontal surface of underlying flexible foundation rock idealized as a viscoelastic half-plane, and impounding a reservoir of water, possibly with alluvium and sediments at the bottom (Figure 2.1). A complete description of the dam-water-foundation system is presented in Fenves and Chopra [1984b, 1985a].

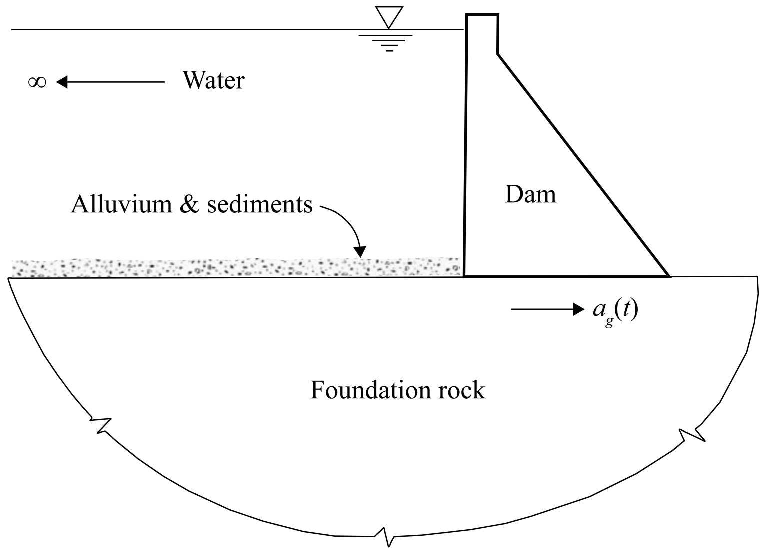


Figure 2.1 Dam-water-foundation system.

2.1 EQUIVALENT STATIC LATERAL FORCES: FUNDAMENTAL MODE

The peak response of the dam in its fundamental vibration mode including dam-water-foundation interaction effects can be estimated by static analysis of the dam alone subjected to equivalent static lateral forces acting on the upstream face of the dam:

$$f_1(y) = \tilde{\Gamma}_1 \frac{A\left(\tilde{T}_1, \tilde{\zeta}_1\right)}{g}\left[w_s(y)\phi_1(y) + gp\left(y, \tilde{T}_r\right)\right] \tag{2.1}$$

in which $\phi_1(y)$ is the horizontal component of displacement at the upstream face of the dam in the fundamental vibration mode shape of the dam supported on rigid foundation with empty reservoir; $w_s(y)$ is the weight per unit height of the dam; and $\tilde{\Gamma}_1 = \tilde{L}_1 / \tilde{M}_1$, where $\tilde{M}_1$ and $\tilde{L}_1$ are given by

$$\tilde{M}_1 = M_1 + \int_0^H p\left(y, \tilde{T}_r\right)\phi_1(y)\,dy \tag{2.2}$$

$$\tilde{L}_1 = L_1 + \int_0^H p\left(y, \tilde{T}_r\right)dy \tag{2.3}$$

in which H is the depth of the impounded water; the generalized mass and earthquake force coefficient are given by

$$M_1 = \frac{1}{g}\int_0^{H_s} w_s(y)\,\phi_1^2(y)\,dy \tag{2.4}$$

$$L_1 = \frac{1}{g}\int_0^{H_s} w_s(y)\,\phi_1(y)\,dy \tag{2.5}$$

where H_s is the height of the dam; g is the acceleration due to gravity; and $A(\tilde{T}_1,\tilde{\zeta}_1)$ is the pseudo-acceleration ordinate of the earthquake design spectrum evaluated at vibration period $\tilde{T}_1$ and damping ratio $\tilde{\zeta}_1$ of the equivalent SDF system representing the dam-water-foundation system.

The function $p(y,\tilde{T}_r)$ is the real-valued component of the complex-valued function representing the hydrodynamic pressure on the upstream face due to harmonic acceleration at period $\tilde{T}_r$ in the shape of the fundamental mode; the corresponding boundary value problem is shown in Figure 2.2a. The natural vibration period of the equivalent SDF system representing the fundamental mode response of the dam (on rigid foundation) with impounded water is given by [Fenves and Chopra 1985a]

$$\tilde{T}_r = R_r T_1 \tag{2.6}$$

in which T_1 is the fundamental vibration period of the dam on rigid foundation with empty reservoir. Hydrodynamic effects lengthen the vibration period, i.e., the period-lengthening ratio, R_r, is greater than one because of the frequency-dependent, added hydrodynamic mass arising from dam-water interaction. It depends on the properties of the dam, the depth of the water, and the absorptiveness of the reservoir bottom materials.

The natural vibration period of the equivalent SDF system representing the fundamental mode response of the dam (with empty reservoir) on flexible foundation is given by [Fenves and Chopra 1985a]

$$\tilde{T}_f = R_f T_1 \tag{2.7}$$

Dam-foundation interaction lengthens the vibration period, i.e., the period-lengthening ratio, R_f, is greater than one because of the frequency-dependent, added foundation flexibility arising from dam-foundation interaction. It depends on the properties of the dam and foundation, most importantly, on the ratio E_f/E_s of the elastic moduli of the foundation and the dam concrete.

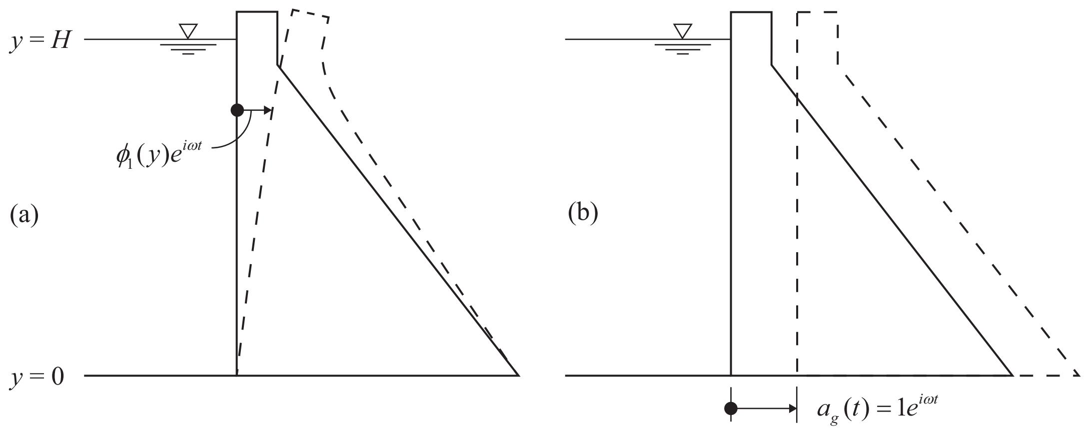


Figure 2.2 **(a) Acceleration of a dam in its fundamental mode shape; (b) horizontal acceleration of a rigid dam.**

The natural vibration period of the equivalent SDF system representing the fundamental mode response of the dam including dam-water-foundation interaction is given by [Fenves and Chopra 1985b]

$$\tilde{T}_1 = R_r R_f T_1 \tag{2.8}$$

The damping ratio of this equivalent SDF system can be expressed as [Fenves and Chopra 1985b]

$$\tilde{\zeta}_1 = \frac{1}{R_r} \frac{1}{(R_f)^3} \zeta_1 + \zeta_r + \zeta_f \tag{2.9}$$

in which ζ_1 is the damping ratio of the dam on rigid foundation with empty reservoir; ζ_r is the added damping due to dam-water interaction and reservoir bottom absorption; and ζ_f is the added radiation and material damping due to dam-foundation interaction. Considering that $R_r > 1$ and $R_f > 1$, Equation (2.9) shows that dam-water interaction and dam-foundation interaction reduce the effectiveness of structural (dam) damping. However, usually this reduction is more than compensated by (a) added damping due to reservoir bottom absorption and (b) dam-foundation interaction, which leads to an increase in the overall damping of the dam.

Before closing this section, we note that the equivalent static lateral forces $f_1(x, y)$ vary over the cross section of the dam monolith. These were integrated over the breadth of the monolith to obtain the forces per unit height of the dam, see Equation (2.1). The variation of the fundamental mode shape $\phi_1^x(x, y)$ over the breadth of the dam is thus neglected, i.e., $\phi_1^x(x, y) \approx \phi_1^x(0, y)$, and the fundamental mode shape at the upstream face of the dam, $\phi_1(y) \equiv \phi_1^x(0, y)$, is used in all subsequent calculations. The implication of the one-dimensional formulation of lateral forces to the estimation of stresses is discussed in Chapter 6.

2.2 EQUIVALENT STATIC LATERAL FORCES: HIGHER MODES

Although the fundamental vibration mode is dominant in the response of the dam, the contributions of the higher modes are included by approximating them using the "static correction" concept [Chopra 2012: Section 12.12 and 13.1.5]. This implies that the ordinates of the pseudo-acceleration design spectrum at the higher mode periods are approximated by the zero-period ordinate, i.e., the peak ground acceleration. The quality of this approximation depends on dynamic amplification of the design spectrum at the higher mode periods, as will be discussed in Chapter 6.

Just as in the case of multistory buildings [Veletsos 1977], soil-structure (dam-foundation) interaction effects may be neglected in a simplified procedure to compute the contributions of the higher vibration modes to the earthquake response of dams.

Utilizing the preceding concepts, the equivalent lateral earthquake forces associated with the higher vibration modes of dams, including the effects of the impounded water, are given by [Fenves and Chopra 1987]

$$f_{\text{sc}}(y) = \frac{a_g}{g}\left\{ w_s(y)\left[1 - \frac{L_1}{M_1}\phi_1(y)\right] + \left[g p_0(y) - \frac{B_1}{M_1} w_s(y)\phi_1(y)\right]\right\} \tag{2.10}$$

In Equation (2.10), a_g is the peak ground acceleration; $p_0(y)$ is a real-valued frequency-independent function for hydrodynamic pressure on a rigid dam undergoing unit acceleration, with water compressibility neglected (Figure 2.2b) (both assumptions being consistent with the "static correction" concept); and B_1 provides a measure of the portion of $p_0(y)$ that acts in the fundamental vibration mode:

$$B_1 = 0.20 \frac{F_{\text{st}}}{g}\left(\frac{H}{H_s}\right)^2 \tag{2.11}$$

where F_{st} is the total hydrostatic force on the dam. The shape of only the fundamental vibration mode enters into Equation (2.10) and the higher mode shapes are not required, thus simplifying the analysis considerably.

2.3 RESPONSE ANALYSIS

As shown in the preceding two sections, the maximum effects of earthquake ground motion in the fundamental vibration mode of the dam have been represented by equivalent static lateral forces $f_1(y)$ and those due to all the higher modes by $f_{\text{sc}}(y)$, determined directly from the response (or design) spectrum without any response history analyses. Static analysis of the dam alone for these two sets of forces provide estimates of the peak modal responses r_1 and r_{sc} for any response quantity, r, e.g., the shear force or bending moment at any horizontal section, or the shear stress or vertical stress at any point. The total response is given by

$$r_{\text{max}} = r_{\text{st}} \pm \sqrt{\left(r_1\right)^2 + \left(r_{\text{sc}}\right)^2} \tag{2.12}$$

where the initial value, r_{st}, of the response quantity prior to the earthquake is determined by standard static analysis procedures, including the effects of the self-weight of the dam, hydrostatic pressures, construction sequence, and thermal effects.

In Equation (2.12) the dynamic response is obtained by combining peak modal responses r_1 and r_{sc} in the fundamental and higher modes, respectively, by the SRSS rule, which is appropriate because the natural vibration frequencies of a concrete gravity dam are well separated. Because the directions of earthquake responses are reversible, both positive and negative signs are included in the dynamic response.

The SRSS combination rule is applicable to the computation of any response quantity that is proportional to the modal coordinates [Chopra 2012: Section 13.8]. Thus, this rule is generally not valid to determine the principal stresses. However, the maximum principal stresses at the two faces of the dam can be determined by a simple transformation of the vertical stresses—determined by beam theory—if the upstream face is nearly vertical and the effects of tail-water at the downstream face are small [Fenves and Chopra 1986: Appendix C]. Under these restricted conditions, the resulting principal stresses at the two faces of a dam monolith (*not in the interior*) may be determined by the SRSS rule.

The preceding combination of static and dynamic responses is appropriate if r_{st}, r_1, and r_{sc} are oriented similarly. Such is obviously the case for the shear and vertical stresses at any point, but generally not for principal stresses except under the restricted conditions previously mentioned.

3 Standard System Properties for Fundamental Mode Response

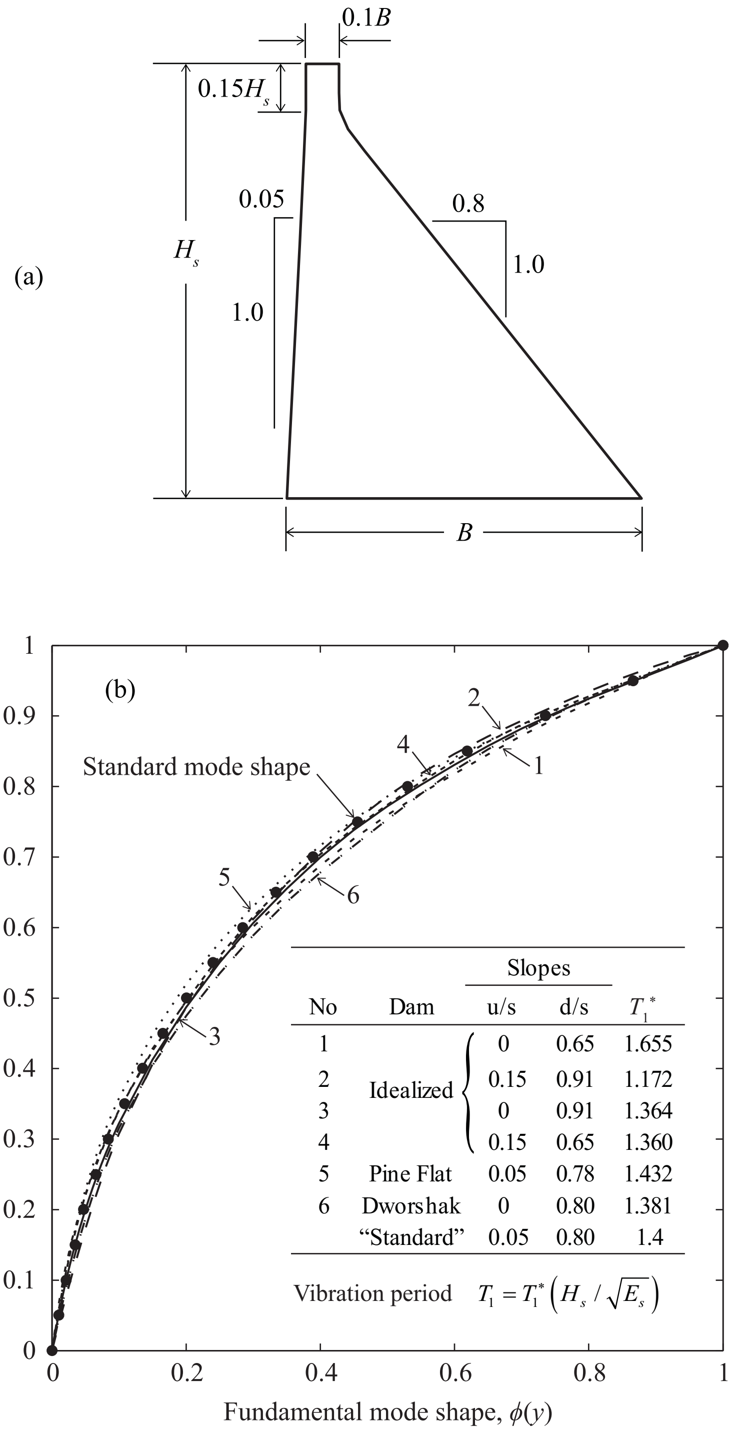
The computations required to directly evaluate Equation (2.1) would be excessive in practical application. Recognizing that the cross-sectional geometry of concrete gravity dams does not vary widely, standard values for the vibration properties—vibration period and shape of the fundamental mode—of the dam, period lengthening ratios R_r and R_f due to dam-water and dam-foundation interaction, damping ratios ζ_r and ζ_f associated with the two interaction mechanisms, and the hydrodynamic pressure functions $p(y,\tilde{T}_r)$ and $p_0(y)$ are presented in this chapter. They represent an extension of the data first presented in Fenves and Chopra [1986].

3.1 VIBRATION PROPERTIES FOR THE DAM

The fundamental vibration period, in seconds, for a "standard" cross section (Figure 3.1a) for non-overflow monoliths of concrete gravity dams on rigid foundation with an empty reservoir can be approximated by [Chopra 1978]

$$T_1 = 1.4\frac{H_s}{\sqrt{E_s}} \tag{3.1}$$

where H_s is the height of the dam in feet, and E_s is the modulus of elasticity of the dam concrete in psi. The fundamental vibration mode shape, $\phi_1(y)$, of the "standard" cross section is shown in Figure 3.1b and presented in Table A.1. These standard vibration properties are compared in Figure 3.1b with the fundamental vibration periods and mode shapes determined by finite element analyses of six cross sections—two actual dams and four idealized dams—chosen to cover the plausible range of shapes. This comparison demonstrates that it is appropriate to use the standard vibration period and mode shape for preliminary design and safety evaluation of concrete gravity dams.



No	Dam	Slopes u/s	Slopes d/s	T_1^*
1	Idealized	0	0.65	1.655
2	Idealized	0.15	0.91	1.172
3	Idealized	0	0.91	1.364
4	Idealized	0.15	0.65	1.360
5	Pine Flat	0.05	0.78	1.432
6	Dworshak	0	0.80	1.381
	"Standard"	0.05	0.80	1.4

Figure 3.1 **(a) "Standard" cross-section; (b) comparison of fundamental vibration period and mode shape for the "standard" cross-section and four idealized and two actual concrete gravity dam cross-sections. Data from Chopra [1978].**

3.2 MODIFICATION OF PERIOD AND DAMPING DUE TO DAM-WATER INTERACTION

Dam-water interaction and reservoir bottom absorption modify the natural vibration period and damping ratio of the equivalent SDF system. For the "standard" dam cross section, the period lengthening ratio R_r and added damping ζ_r are dependent on several parameters, the most significant being: modulus of elasticity E_s of the dam concrete, the ratio H/H_s of water depth to dam height, and the wave reflection coefficient α. This coefficient, α, is the ratio of the amplitude of the reflected hydrodynamic pressure wave to the amplitude of a vertically propagating pressure wave incident on the reservoir bottom [Fenves and Chopra 1983, 1984b], where $\alpha = 1$ indicates complete reflection of pressure waves, and smaller values of α indicate increasingly absorptive materials.

By performing many analyses of the "standard" dam cross section using the procedures described in Fenves and Chopra (1984a) and modified in Appendix A of Fenves and Chopra (1986) for dams with large values of modulus of elasticity E_s, period lengthening ratio R_r and added damping ratio ζ_r have been computed as a function of H/H_s for a range of values of E_s and α [Fenves and Chopra 1986]; results are summarized in Table A.2.

The mechanics of dam-water interaction and reservoir bottom absorption has been discussed elsewhere in detail [Fenves and Chopra 1983, 1984b]. Here, we simply note that R_r increases and ζ_r generally—but not always—increases, with increasing water depth, absorptiveness of the reservoir bottom materials, and elastic modulus of concrete. The effects of dam-water interaction may be neglected in the analysis if the reservoir depth is less than half of the dam height, i.e., $H/H_s < 0.5$.

3.3 MODIFICATION OF PERIOD AND DAMPING DUE TO DAM-FOUNDATION INTERACTION

Dam-foundation interaction modifies the natural vibration period and damping ratio of the equivalent SDF system. For the "standard" dam cross section, period lengthening ratio R_f and added damping ζ_f depend on several parameters, the most significant being: E_f/E_s, the ratio of the moduli of elasticity of the foundation rock to that of the dam concrete; and η_f, the constant hysteretic damping factor for the foundation rock.

By performing many analyses of the "standard" dam cross section using the procedures described in Fenves and Chopra [1984a], period lengthening ratio R_f and added damping ratio ζ_f were initially computed for a range of values of E_f/E_s and η_f = 0.01, 0.10, 0.25, and 0.50 [Fenves and Chopra 1986], which in retrospect turned out to be too coarse. The added damping ratio has now been recomputed for a closely spaced set of η_f values; the results are presented in Table A.3.

The mechanics of dam-foundation interaction has been discussed elsewhere in detail [Fenves and Chopra 1984b]. Here we simply note that for moduli ratios E_f/E_s that are representative of actual dam sites, the period ratio R_f varies little with η_f; therefore a single curve represents the variation of R_f with E_f/E_s, which may be used for any value of η_f. As expected, R_f increases as the moduli ratio E_f/E_s decreases, which for a fixed value of E_s implies that the foundation is increasingly flexible. The added damping ratio ζ_f increases with decreasing E_f/E_s and increasing constant hysteretic damping factor η_f. The foundation may be treated as rigid in the analysis if $E_f/E_s > 4$, as the effects of dam-foundation interaction are then negligible.

3.4 HYDRODYNAMIC PRESSURE

In order to provide a convenient means for determining the hydrodynamic pressure function $p(y,\tilde{T}_r)$ in Equation (2.1), a non-dimensional form of this function, $gp(\hat{y})/wH$, where $\hat{y} = y/H$ and w is the unit weight of water, was computed in Fenves and Chopra (1986) for several values of α and a range of the period ratio

$$R_w = \frac{T_1^r}{\tilde{T}_r} \tag{3.2}$$

where T_1^r is the fundamental vibration period of the impounded water given by $T_1^r = 4H/C$, where C is the velocity of pressure waves in the water. Results for a full reservoir, $H/H_s = 1$, and a range of values of α and R_w are summarized in Table A.4. The function $gp(\hat{y})/wH$ for other values of H/H_s can be approximately computed as $(H/H_s)^2$ times the function for $H/H_s = 1$ [Chopra 1978].

3.5 GENERALIZED MASS AND EARTHQUAKE FORCE COEFFICIENT

Instead of evaluating Equations (2.2) and (2.3), the generalized mass, $\tilde{M}_1$, and generalized earthquake coefficient, $\tilde{L}_1$, of the equivalent SDF system including hydrodynamic effects can be conveniently computed from [Fenves and Chopra 1986]

$$\tilde{M}_1 = (R_r)^2 M_1 \tag{3.3}$$

$$\tilde{L}_1 = L_1 + \frac{1}{g} F_{\text{st}} \left(\frac{H}{H_s} \right)^2 A_p \tag{3.4}$$

where $F_{\text{st}} = wH^2/2$ is the hydrostatic force, and the hydrodynamic force coefficient A_p is the integral over the depth of water of the pressure function $2gp(\hat{y})/wH$ for $H/H_s = 1$. The hydrodynamic force coefficient, A_p, computed in Fenves and Chopra [1986] for a range of values for period ratio R_w and wave reflection coefficient α, are summarized in Table A.5.

4 Implementation of Analysis Procedure

4.1 SELECTION OF SYSTEM PARAMETERS AND EARTHQUAKE DESIGN SPECTRUM

The response spectrum analysis (RSA) procedure requires only a few parameters to describe the dam-water-foundation system: E_s, ζ_1, H_s, E_f, η_f, H, and α. In addition, a pseudo-acceleration design spectrum is required to represent the seismic hazard at the site. Based on the recommendations presented in Fenves and Chopra [1987], with a few modifications, guidelines for selecting the system parameters to be used in the RSA procedure are presented in this section.

The Young's modulus of elasticity E_s for the dam concrete should be based on suitable test data—in so as far as possible—or estimated from the design strength of concrete. The value of E_s may be modified to recognize the strain rates representative of those the concrete may experience during earthquake motions of the dam [Chopra 1978]. The dam-water interaction parameters R_r and ζ_r may be estimated for the selected E_s value by linearly interpolating, if necessary, between the nearest values for which data are available in Table A.2: E_s = 1.0, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, or 5.0 million psi. Correlation of recorded and computed motions of dams during earthquakes [Chopra and Wang 2010], indicates that the viscous damping ratio ζ_1 for the dam alone is in the range of 1 to 3%. Assigning a value for ζ_1 in this range is recommended if no data specific to the dam is available. The height H_s of the dam is measured from the base to the crest.

The Young's modulus of elasticity E_f and constant hysteretic damping coefficient η_f of the foundation rock should be determined from a site investigation and appropriate tests. For the resulting value of E_f / E_s, the dam-foundation interaction parameters R_f and ζ_f can be estimated by linearly interpolating, if necessary, between the two nearest values for which data are available in Table A.3. In the absence of measured properties for the rock at the site, a value of η_f in the range of 0.02–0.06 is recommended [Chopra and Wang 2010], corresponding to a viscous damping ratio of 1–3%.

The depth H of the impounded water is measured from the free surface to the reservoir bottom. In practical situations the elevations of the reservoir bottom and dam base may differ. The standard values for unit weight of water and velocity of pressure waves in water are $w = 62.4$ pcf and $C = 4720$ ft/sec, respectively.

It may be impractical to determine reliably the wave reflection coefficient α because the reservoir bottom materials may consist of highly variable layers of exposed bedrock, alluvium, silt, and other sediments, and appropriate site investigation techniques have not been developed. However, to be conservative, the estimated value of α should be rounded up to the nearest value for which data are presented: α = 1.0, 0.90, 0.75, 0.50, 0.25, and 0; interpolation of data for intermediate values of α is not appropriate. For proposed new dams or recent dams where sediment deposits are meager, α = 0.90 or 1.0 is recommended and, lacking data, α = 0.75 or 0.90 is recommended for older dams where sediment deposits are substantial. In each case, the larger α value will generally give conservative results, which is appropriate at the preliminary design stage.

The horizontal earthquake ground acceleration is specified by a pseudo-acceleration design spectrum in the RSA procedure. This should be a smooth response spectrum—without the irregularities inherent in response spectra of individual ground motions—representative of the intensity and frequency characteristics of the earthquake events associated with the seismic hazard at the site.

4.2 COMPUTATIONAL STEPS

Computation of the earthquake response of the dam is organized in three parts [Fenves and Chopra 1987]:

Part I: Compute the earthquake forces and stresses due to response of the dam in its fundamental mode of vibration by the following computational steps:

l. Compute T_1, the fundamental vibration period of the dam, in seconds, on rigid foundation with an empty reservoir from Equation (3.1) in which H_s is the height of the dam in feet, and E_s is the design value of the modulus of elasticity of dam concrete in psi.

2. Compute $\tilde{T}_r$, the fundamental vibration period of the dam, in seconds, including the influence of impounded water from Equation (2.6) in which T_1 was computed in Step 1; R_r is the period ratio determined from Table A.2 for the design values of E_s, the wave reflection coefficient α, and the depth ratio H/H_s, where H is the depth of the impounded water. If $H/H_s < 0.5$, computation of R_r may be avoided by using $R_r = 1$.

3. Compute the period ratio R_w from Equation (3.2) in which $\tilde{T}_r$ was computed in Step 2; and $T_1^r = 4H/C$ where C = 4720 ft/sec.

4. Compute $\tilde{T}_1$, the fundamental vibration period of the dam, in seconds, including the dam-water-foundation interaction, from Equation (2.8) in which R_r was determined in Step 2; R_f is the period ratio determined from Table A.3 for the design value of E_f/E_s; and E_f is the modulus of elasticity of the foundation. If $E_f/E_s > 4$, use $R_f \approx 1$.

5. Compute the damping ratio $\tilde{\zeta}_1$ of the dam from Equation (2.9) using the computed period ratios R_r and R_f; ζ_1 is the viscous damping ratio for the dam on rigid foundation with empty reservoir; ζ_r is the added damping ratio due to dam-water interaction and reservoir bottom absorption, obtained from Table A.2 for the selected values of E_s, α and H/H_s; ζ_f is the added damping ratio due to dam-foundation interaction, obtained from Table A.3 for the selected values of E_f/E_s, and η_f. If $H/H_s < 0.5$, use $\zeta_r = 0$; if $E_f/E_s > 4$, use $\zeta_f = 0$; and if the computed value of $\tilde{\zeta}_1 < \zeta_1$, use $\tilde{\zeta}_1 = \zeta_1$.

6. Determine $gp(y,\tilde{T}_r)$ from Table A.4 corresponding to the value of R_w computed in Step 3 (by interpolating, if necessary, between data for the two nearest available values of R_w), the design value of α, and for $H/H_s = 1$; the result is multiplied by $(H/H_s)^2$. If $H/H_s < 0.5$, computation of $p(y,\tilde{T}_r)$ may be avoided by using $p(y,\tilde{T}_r) \approx 0$.

7. Compute the generalized mass, $\tilde{M}_1$, from Equation (3.3) in which R_r was computed in Step 2; and M_1 is computed from Equation (2.4) in which $w_s(y)$ is the weight of the dam per unit height; the fundamental vibration mode shape $\phi_1(y)$ is tabulated in Table A.1; and g is the acceleration due to gravity.

8. Compute the generalized earthquake force coefficient $\tilde{L}_1$ from Equation (3.4) in which L_1 is computed from Equation (2.5); $F_{\text{st}} = wH^2/2$; and A_p is given in Table A.5 for the values of R_w and α used in Step 6. If $H/H_s < 0.5$, computation of $\tilde{L}_1$ may be avoided by using $\tilde{L}_1 \approx L_1$.

9. Compute $f_1(y)$, the equivalent static lateral earthquake forces associated with the fundamental vibration mode from Equation (2.1) in which $A(\tilde{T}_1,\tilde{\zeta}_1)$ is the pseudo-acceleration ordinate of the earthquake design spectrum evaluated at the vibration period $\tilde{T}_1$ determined in Step 4 and damping ratio $\tilde{\zeta}_1$ determined in Step 5; $w_s(y)$ is the weight per unit height of the dam; $\phi_1(y)$ is the fundamental vibration mode shape of the dam from Table A.1; $\tilde{\Gamma}_1 = \tilde{L}_1/\tilde{M}_1$ where $\tilde{L}_1$ and $\tilde{M}_1$ was determined in Steps 7 and 8, respectively; and the hydrodynamic pressure term $gp(y,\tilde{T}_r)$ was determined in Step 6.

10. Determine by static analysis of the dam subjected to the equivalent static lateral forces $f_1(y)$, from Step 9, applied to the upstream face of the dam, all the response quantities of interest, in particular, the stresses throughout the dam. Traditional procedures for design calculations may be used wherein the bending stresses across a horizontal section are computed by elementary formulas for stresses in beams. Alternatively, the finite element method may be used for a more accurate static stress analysis.

    Note: If computed using beam theory, stresses at the sloping part of the downstream face should be multiplied by the correction factor of 0.75 developed in Section 4.3.

Part II: The earthquake forces and stresses due to the higher vibration modes can be determined approximately for purposes of preliminary design by the following computational steps:

11. Compute $f_{sc}(y)$, the equivalent static lateral earthquake forces associated with the higher vibration modes from Equation (2.10) in which M_1 and L_1 were determined in Steps 7 and 8, respectively; $gp_0(y)$ is determined from Table A.6; B_1 is computed from Equation (2.11); and a_g is the peak ground acceleration from the earthquake design spectrum. If $H/H_s < 0.5$, computation of $p_0(y)$ may be avoided by using $p_0(y) \approx 0$ and hence $B_1 \approx 0$.

12. Determine by static analysis of the dam subjected to the equivalent static lateral forces $f_{sc}(y)$, from Step 11, applied to the upstream face of the dam, all the response quantities of interest, in particular, the stresses throughout the dam. The stress analysis may be carried out by the same procedures mentioned in Step 10.

Part III: The total bending moments, shear forces and stresses at any section in the dam are determined by the following computational step:

13. Compute the total value of any response quantity from Equation (2.12) in which r_1 and r_{sc} are values of the response quantity determined in Steps 10 and 12 associated with the fundamental and higher vibration modes, respectively; and r_{st} is its initial value prior to the earthquake due to various loads, including the self-weight of the dam, hydrostatic pressure, construction sequence, and thermal effects.

4.3 CORRECTION FACTOR FOR DOWNSTREAM FACE STRESSES

Formulas based on beam theory overestimate stresses at sloping faces, thus, stresses computed at the downstream face of concrete gravity dams should be multiplied by the correction factor developed in this section.

Figure 4.1 shows the vertical stresses, $\sigma_{y,1}$, at the upstream and downstream faces of Pine Flat Dam (Figure 6.1), which is typical of many dams, with empty reservoir on rigid foundation, due to the lateral forces of Equation (2.1). Stresses were computed by static analysis using beam formulas and the finite element method; a detailed summary of the procedure is included in Appendix C. It is evident that beam theory provides results close to those from finite element analysis at the upstream face, but the stresses at the downstream face are considerably overestimated. Multiplying the stress values at the sloping part of the downstream face by a correction factor of 0.75 leads to stresses that are much closer to the finite element values. However, the agreement is not as good near the toe of the dam and at the stress concentration where the downstream face changes slope.

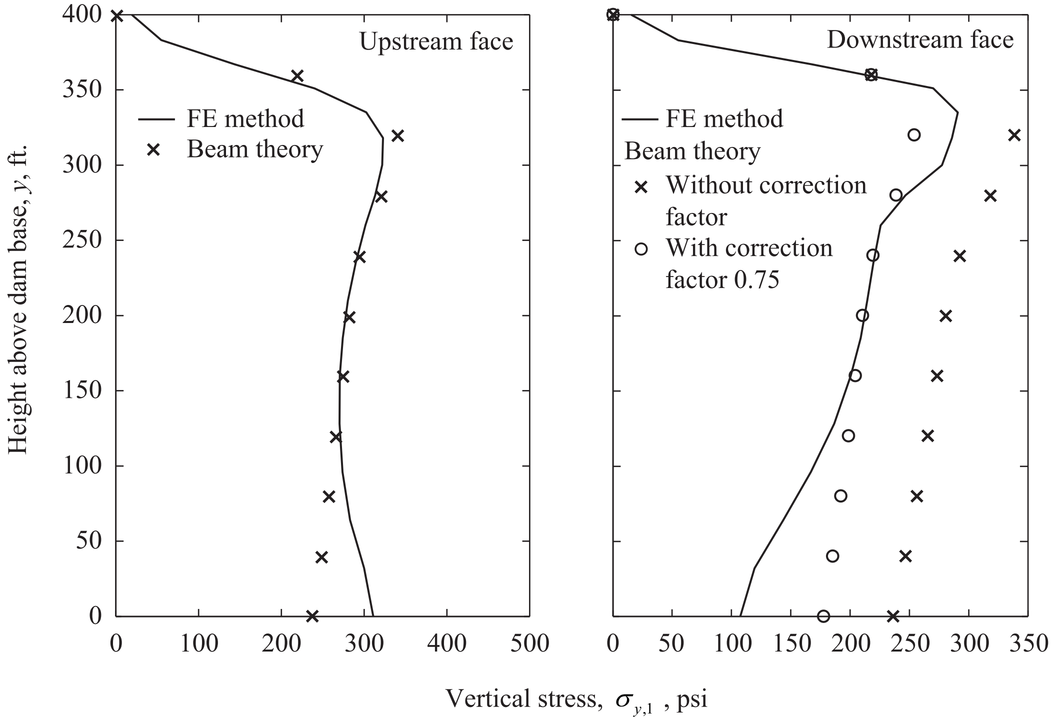


Figure 4.1 Vertical stresses, $\sigma_{y,1}$, at the upstream and downstream face of Pine Flat Dam with empty reservoir on rigid foundation due to the lateral forces of Equation (2.1).

The correction factor of 0.75 is applicable for modifying vertical stresses computed by beam theory if the slope of the downstream face is no steeper than 0.8:1; it will give conservative results for flatter slopes, but will underestimate the stresses if the slope is much steeper than 0.8:1. The same correction factor is applicable to the principal stresses computed by beam theory at the downstream face of the dam provided the stresses due to tail-water are negligible. With this restriction, the principal stresses are directly proportional to the vertical stresses [Fenves and Chopra 1986].

Although the correction factor was determined from computed stresses due to the lateral forces associated with the fundamental mode only, it may also be applied to the higher mode stresses, $\sigma_{y,sc}$. The effectiveness of the correction factor applied to both modal contributions is demonstrated in Section 6.3.

4.4 USE OF S.I. UNITS

Because the standard values for most quantities required in the RSA procedure are presented in a non-dimensional form, implementation of the procedure using S.I. units is straightforward. The expressions and data requiring conversion to S.I. units are noted here:

1. The fundamental vibration period T_1 of the dam on rigid foundation with empty reservoir (Step l), in seconds, is given by:

$$T_1 = 0.38 \frac{H_s}{\sqrt{E_s}} \tag{4.1}$$

   where H_s is the height of the dam in meters; and E_s is the modulus of elasticity of the dam concrete in MPa.

2. The period ratio R_r and added damping ratio ζ_r due to dam-water interaction presented in Table A.2 is for specified values of E_s in psi, which should be converted to MPa as follows: 1 million psi $\approx$ 7 thousand MPa.

3. Where required in the calculations, the unit weight of water $w = 9.81\,\text{kN/m}^3$, the acceleration due to gravity $g = 9.81\,\text{m/s}^2$, and velocity of pressure waves in water $C = 1440$ m/sec.

5 CADAM Computer Program

CADAM—computer aided stability analysis of gravity dams—is a computer program, freely available, developed at the École Polytechnique de Montréal, Canada for static and seismic stability evaluations of concrete gravity dams [Leclerc, Legér and Tinawi 2003]. A screenshot of the user interface is shown in Figure 5.1. Based on the gravity method, CADAM uses rigid body equilibrium and beam theory to perform stress analyses and compute crack lengths and safety factors for dams subjected to various static and seismic load cases (listed in Figure 5.2); a summary of the analyses options available in the program is listed in Table 5.1.

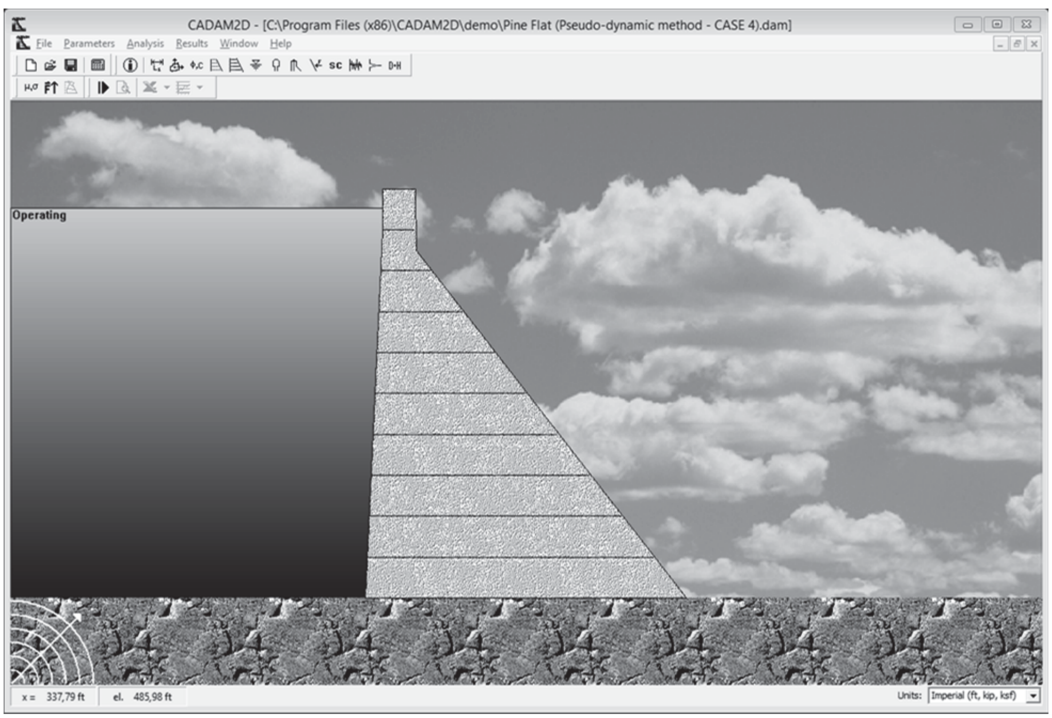


Figure 5.1 Screenshot of CADAM user interface.

Table 5.1 List of analysis options currently available in CADAM [Leclerc, Legér, and Tinawi 2002].

Static analyses	Static analyses are performed for the normal operating reservoir elevation or the flood elevation including overtopping over the crest and floating debris.
Seismic analyses	Seismic analyses are performed using the pseudo-static method (seismic coefficient method) or the pseudo-dynamic method.
Post-seismic analyses	In post-seismic safety analysis, the crack length induced by the seismic event could alter the cohesive shear resistance and uplift pressures. The post-seismic uplift pressures could either (a) build-up to its full value in seismic cracks or (b) return to its initial value if the seismic crack is closed after the earthquake.
Incremental load analyses	Sensitivity analyses are automatically performed by computing and plotting the evolution of typical performance indicators (ex: sliding safety factor) as a function of a progressive application in the applied loading (ex: reservoir elevation, peak ground acceleration).
Probabilistic safety analyses	Probabilistic safety analyses are performed to compute the probability of failure of a dam-foundation-reservoir system as a function of the uncertainties in loading and strength parameters that are considered as random variables with specified probability density functions. A Monte-Carlo simulation computational procedure is used. Static, seismic, as well as post-seismic analyses may be considered.

CADAM implements the RSA procedure, referring to it as the "pseudo-dynamic method." Starting with user input, the program computes the equivalent static lateral forces associated with the response of the system in its fundamental mode and higher vibration modes by implementing the procedure as described in Chapter 4 of this report. The earthquake-induced bending moments, shear forces, and stresses due to the two sets of forces are computed and combined to determine the total dynamic response. Finally, the responses due to earthquake forces and initial static loads can be combined.

The program provides a fully integrated computing environment with output reports and graphical support to visualize input parameters and output performance indicators such as stresses, crack lengths, resultant positions and safety factors. In addition, output can be exported to Microsoft Excel spreadsheets to allow users to perform further post-processing of results.

CADAM is widely used for educational purposes, R&D in dam engineering, and in actual projects. A complete description of the program and its capabilities can be found in Leclerc, Legér, and Tinawi [2003]. The latest [2013] version of CADAM, implementing the standard vibration properties presented in Appendix A, is available for download from:

http://www.polymtl.ca/structures/telecharg/cadam/telechargement.php

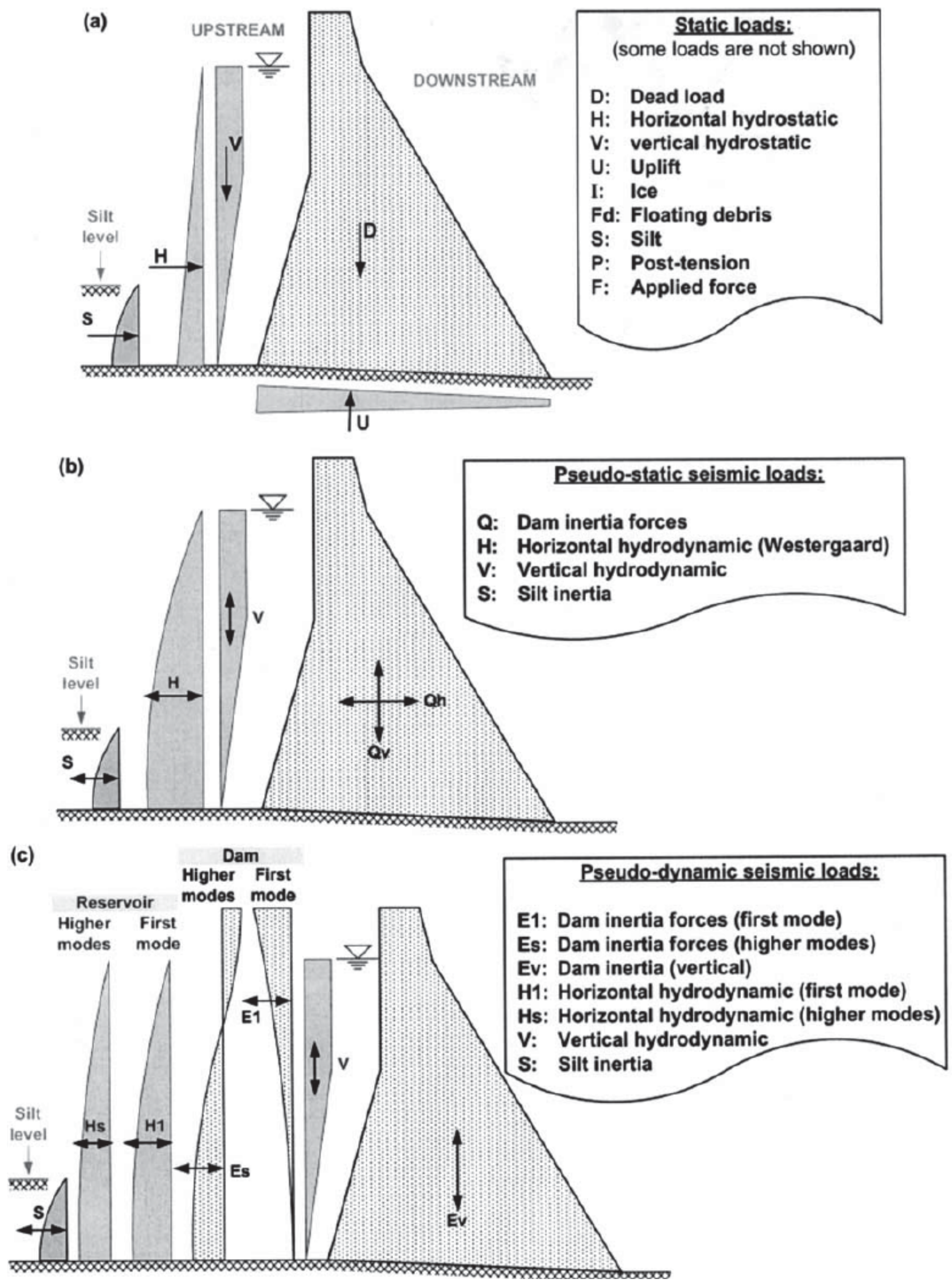


Figure 5.2 **CADAM loading conditions for static and seismic analyses: (a) basic static analysis conditions; (b) pseudo-static seismic analysis; (c) pseudo-dynamic (or RSA) seismic analysis. From Leclerc, Legér, and Tinawi [2003].**

6 Evaluation of Response Spectrum Analysis Procedure

Although based on structural dynamics theory, the RSA procedure involves several approximations which have been checked individually [Fenves and Chopra 1985a, 1985b]. Presented in this chapter is an overall evaluation of the procedure, by comparing its results with those obtained from response history analysis (RHA) of the dam modeled as a finite element system, including dam-water-foundation interaction and reservoir bottom absorption [Fenves and Chopra 1984b]; the later set of results were computed by a newer version of the program EAGD-84 [Fenves and Chopra 1984c].

6.1 SYSTEM CONSIDERED

The system considered is the tallest, non-overflow monolith of Pine Flat Dam shown in Figure 6.1, with the following properties: height of the dam, H_s = 400 ft; modulus of elasticity of concrete, E_s = 3.25 million psi; unit weight of concrete, w_s = 155 pcf; viscous damping ratio for the dam alone, ζ_1 = 2%; modulus of elasticity of the foundation, E_f = 3.25 million psi; constant hysteretic damping factor for the foundation, η_f = 0.04 (corresponding to 2% viscous damping); depth of water, H = 381 ft; and wave reflection coefficient at the reservoir bottom, α = 0.75.

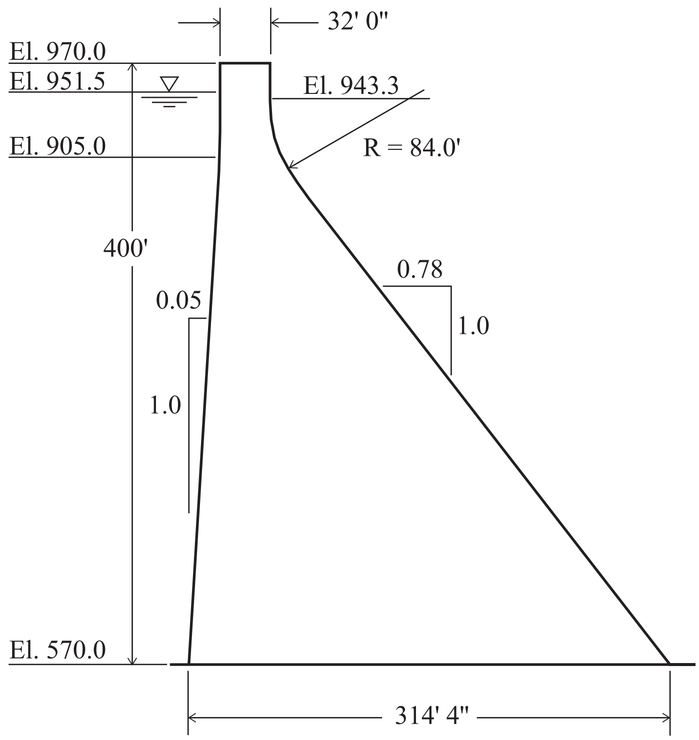


Figure 6.1 Tallest, non-overflow monolith of Pine Flat Dam.

6.2 GROUND MOTIONS

Based on a probabilistic seismic hazard analysis (PSHA) for the Pine Flat Dam site at the 1% in 100 years hazard level, a Conditional Mean Spectrum was developed. A total of 29 ground motion records on rock or NEHERP soil class D or stiffer sites, at a distance R = 0–50 km from earthquakes of magnitude M_w = 5.0–7.5 were selected; the selected range of M_w and R is consistent with the deaggregation of the seismic hazard at the site. Each of the resulting 58 ground motions (two horizontal components of 29 records) was amplitude-scaled to minimize the mean square difference between the response spectrum and the target spectrum over the period range of interest $0.3 \leq T \leq 0.5$ sec. A summary of the PSHA, as well as the selection and scaling of records is presented in Appendix B. The median (computed as the geometric mean) of the response spectra for the 58 ground motions is presented in Figure 6.2.

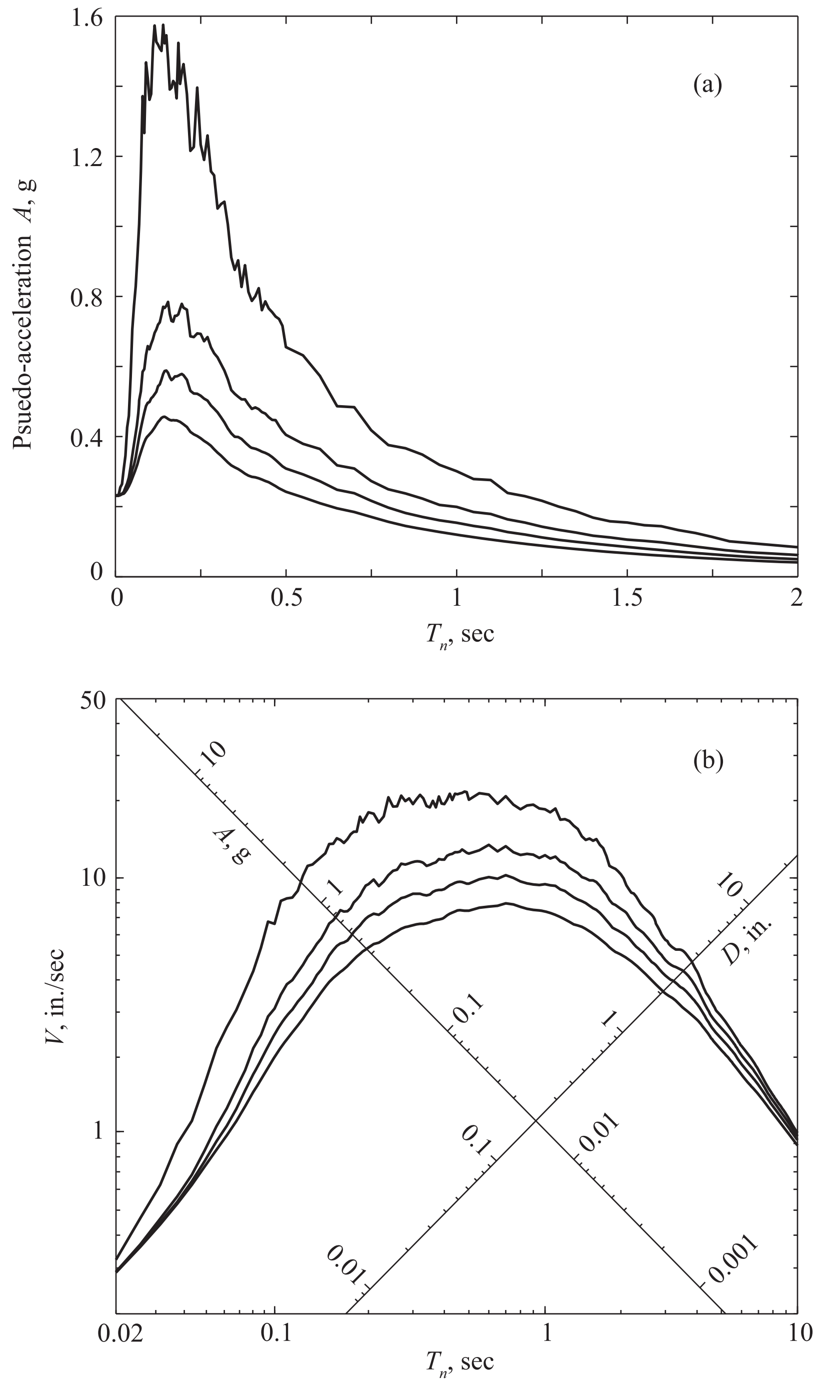


Figure 6.2 **Median response spectra for 58 ground motions: ζ = 0, 2, 5, and 10 percent; (a) linear plot; (b) four-way logarithmic plot.**

6.3 RESPONSE SPECTRUM ANALYSIS

6.3.1 Equivalent Static Lateral Forces

With the earthquake excitation defined by the median response spectrum of Figure 6.2, the dam is analyzed by the RSA procedure for the four cases listed in Table 6.1; for this purpose the step-by-step procedure described in Chapter 4 is implemented (see Appendix C for details). The vibration period and damping ratio of the equivalent SDF system with the corresponding spectral ordinates are presented in Table 6.1, and the equivalent static lateral forces $f_1(y)$ and $f_{sc}(y)$, representing the maximum earthquake effects of the fundamental and higher modes of vibration, respectively, are presented in Figure 6.3.

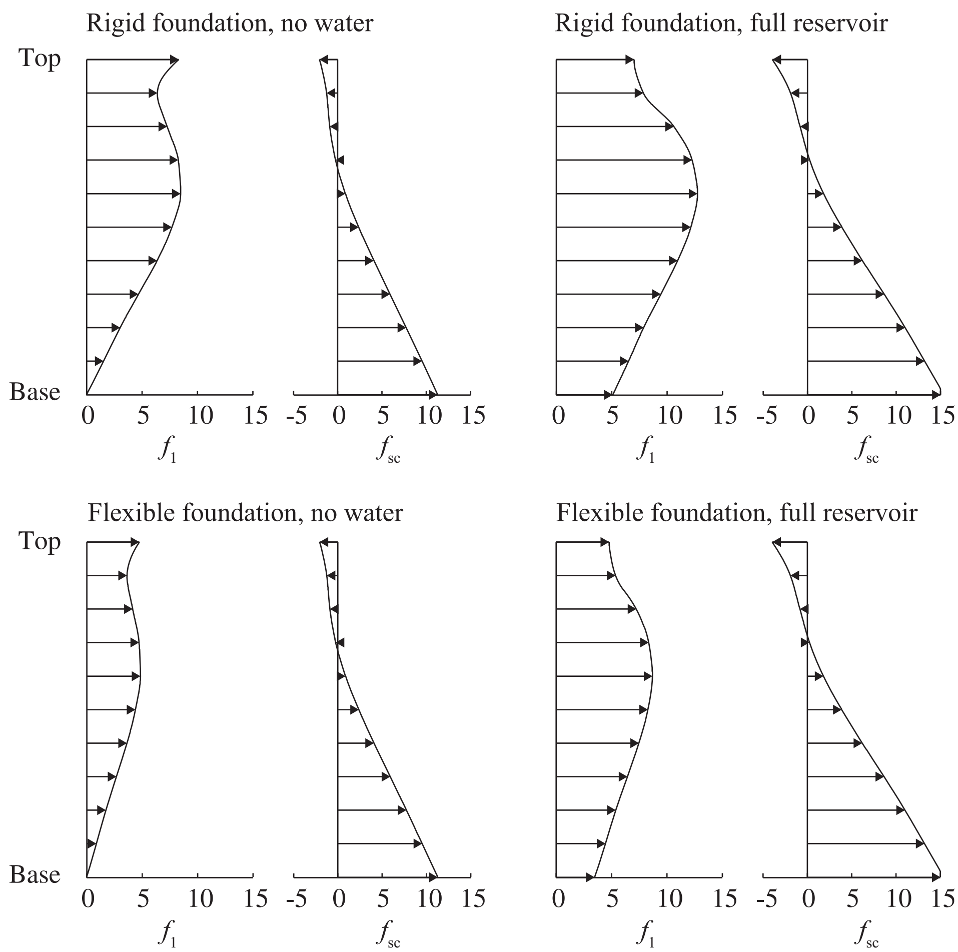


Figure 6.3 Equivalent static lateral forces, f_1 and f_{sc}, on Pine Flat Dam, in kips per foot height, computed by the RSA procedure for four analysis cases.

Table 6.1 **Pine Flat Dam analysis cases, fundamental mode properties, and corresponding pseudo-acceleration ordinates.**

Analysis Case	Foundation	Water	$\tilde{T}_1$, in sec	$\tilde{\zeta}_1$, in percent	$A(\tilde{T}_1, \tilde{\zeta}_1)$, in g
1	Rigid	Empty	0.311	2.0	0.606
2	Rigid	Full	0.387	3.9	0.409
3	Flexible	Empty	0.369	7.1	0.347
4	Flexible	Full	0.459	9.2	0.274

6.3.2 Computation of Stresses

The vertical stresses $\sigma_{y,1}$ and $\sigma_{y,\mathrm{sc}}$ due to the two sets of forces f_1 and f_{sc} are computed by static stress analysis of the dam by two methods: (1) elementary formulas for stresses in beams; and (2) finite element analysis of the dam. Combining $\sigma_{y,1}$ and $\sigma_{y,\mathrm{sc}}$ by the SRSS combination rule leads to the earthquake induced vertical stresses, $\sigma_{y,d}$, presented in Figure 6.4; note that stresses due to initial static loads are not included. The stress values presented occur as tensile stresses at the upstream face when the earthquake forces act in the downstream direction, and at the downstream face when the earthquake forces act in the upstream direction. A detailed description of the computational procedure is included in Appendix C.

The results presented in Figure 6.4 confirm that the correction factor of 0.75 for stresses computed by beam theory at the sloping part of the downstream face is satisfactory for all four cases. The stresses determined by beam theory with the correction factor are very close to those determined by finite element analysis except near the heel and toe of the dam. Therefore, only the stresses from RSA determined by beam theory are compared with the results from RHA in Section 6.4.2.

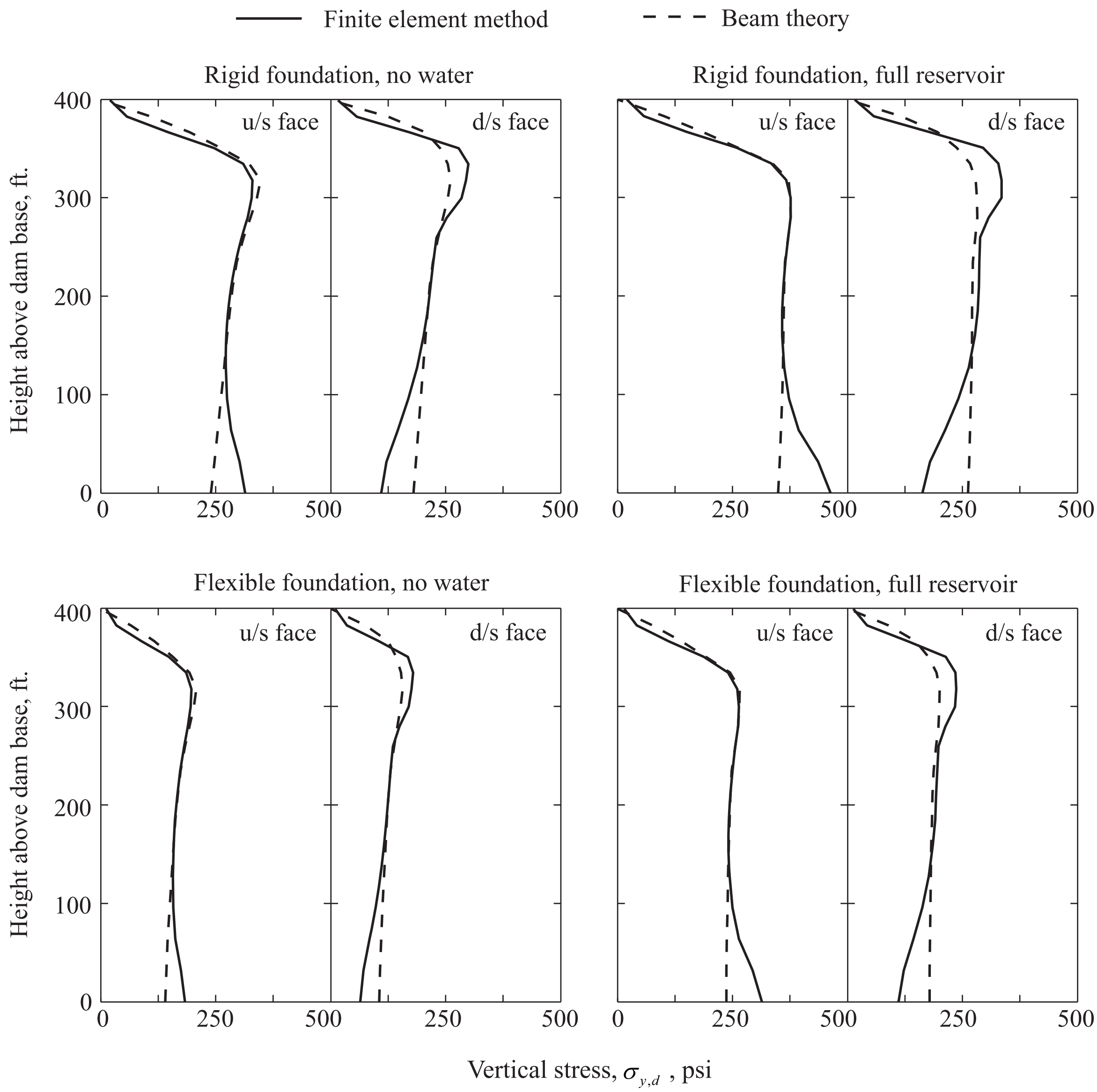


Figure 6.4 **Earthquake induced vertical stresses, $\sigma_{y,d}$, in Pine Flat Dam computed in the RSA procedure by two methods: beam theory and the finite element method.**

6.4 COMPARISON WITH RESPONSE HISTORY ANALYSIS

Response history analysis of the dam monolith modeled as a finite element system, considering rigorously the effects of dam-water-foundation interaction and reservoir bottom absorption, is implemented by a newer version of the computer program EAGD-84 [Fenves and Chopra 1984c] for each of the 58 ground motions. In the following sections, results computed by RSA and RHA procedures are compared.

6.4.1 Fundamental Mode Properties

The fundamental vibration period and the effective damping ratio at this period are estimated using Equations (2.6) - (2.9) in the RSA procedure. These vibration properties are not needed in the RHA procedure; however, for the purposes of evaluating the accuracy of the approximate results, they are determined—by the half-power bandwidth method—from the frequency response function for the fundamental mode response of the dam-water-foundation system computed in the EAGD-84 program. These are referred to as the "exact" results in Table 6.2.

It is apparent that the approximate procedure provides excellent estimates for the resonant period and effective damping ratio of the system in its fundamental mode, confirming that the equivalent SDF model for the dam-water-foundation system is able to represent the important effects of dam-water interaction, reservoir bottom absorption and dam-foundation interaction.

Table 6.2 "Exact" and approximate fundamental mode properties.

			Vibration Period, $\tilde{T}_1$, in sec		Damping Ratio, $\tilde{\zeta}_1$, in percent	
Case	Foundation	Water	Approx.	Exact	Approx.	Exact
1	Rigid	Empty	0.311	0.318	2.0	2.0
2	Rigid	Full	0.387	0.395	3.9	3.2
3	Flexible	Empty	0.369	0.390	7.1	8.7
4	Flexible	Full	0.459	0.491	9.2	9.8

6.4.2 Stresses

The peak value of the maximum principal stress at a location over the duration of each ground motion is determined from the response history computed by the EAGD-84 program, see Appendix C. At the two faces of the dam, the principal stresses are essentially parallel to the faces if the upstream face is nearly vertical and the stresses due to tail-water at the downstream face are negligible [Fenves and Chopra 1986]; these conditions are usually satisfied in practical problems. This implies that the direction of the peak value of maximum principal stress at locations on a dam face is essentially invariant among ground motions, therefore the peak stress values due to the 58 ground motions lend themselves to statistical analysis.

At each location on the two faces of the dam the median value is computed as the geometric mean of the data set; results are presented in Figure 6.5 where they are compared with the RSA results. The maximum principal stresses in the RSA procedure are obtained by a transformation of the vertical stresses determined by beam theory.

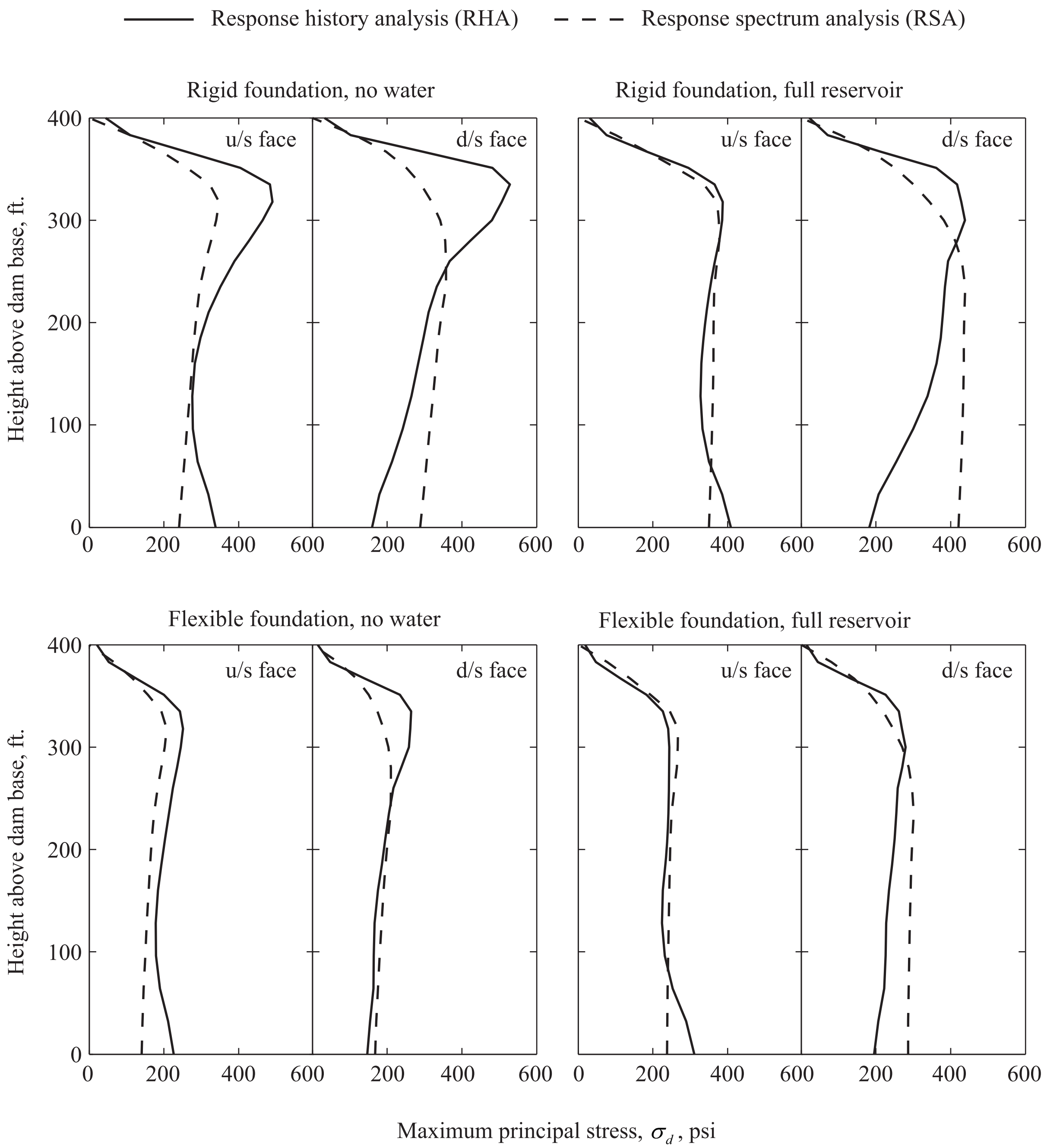


Figure 6.5 **Comparison of peak values of maximum principal stresses in Pine Flat Dam computed by RSA and RHA procedures; initial static stresses are excluded.**

Case 1 (rigid foundation, empty reservoir) is an example where higher mode contributions are considerable, primarily in the upper part of the dam, as expected, where the steep stress gradients are evident in the RHA results (Figure 6.5). The RSA procedure underestimates these higher mode contributions because the vibration periods are not short enough for the static correction approximation to be valid. As shown in Figure 6.6, the spectral accelerations at the second- and third-mode periods are more than three times the peak ground acceleration that is used instead in the static correction method. Thus, the static correction method grossly underestimates the higher mode stresses. For the median response spectrum considered, such discrepancy would be much smaller in the case of a dam of lower height with shorter periods. For Cases 2–4 the RSA procedure provides very good estimates of the maximum principal stresses.

The RSA procedure tends to be more conservative—relative to the RHA results—at the downstream face of the dam than at the upstream face (Figure 6.5). An investigation revealed that the underlying reason is the one-dimensional representation of the equivalent static lateral forces in Equation (2.1), wherein any variation of the fundamental mode shape over the breadth of the dam was neglected, thus ignoring the horizontal variation of the lateral forces.

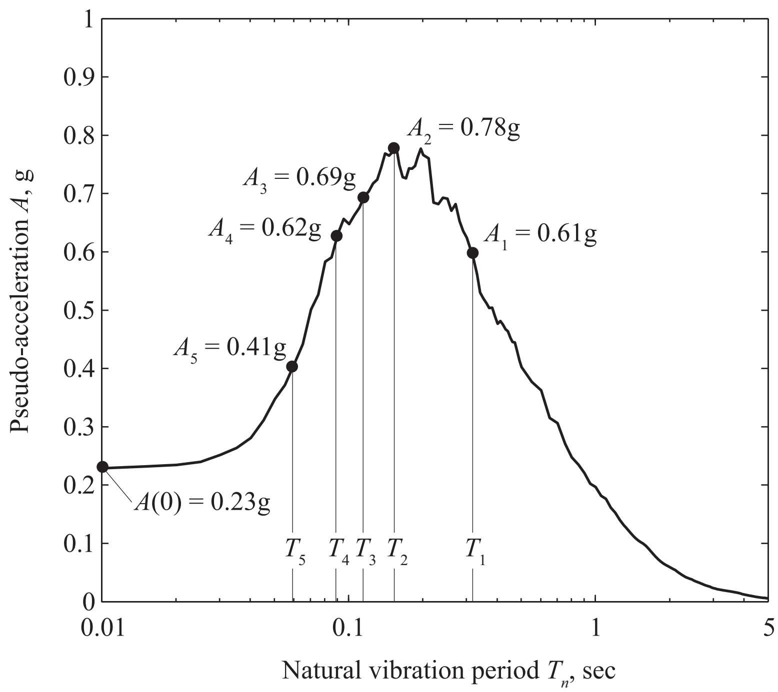


Figure 6.6 **Spectral accelerations at the first five natural vibration periods of Pine Flat Dam on rigid foundation with empty reservoir; damping, ζ = 2%.**

The preceding results demonstrate that the RSA procedure estimates stresses to a degree of accuracy that is satisfactory for the preliminary phase in the design of new dams and in the safety evaluation of existing dams. The level of accuracy achieved in the RSA procedure is noteworthy, especially considering the complicated effects of dam-water-foundation interaction and reservoir bottom absorption on the dynamics of the system, and the number of approximations necessary to develop the procedure. The accuracy of the computed results depends on several factors, including how well the fundamental resonant period and damping ratio are estimated in the RSA procedure, and how well the static correction method is able to account for the contributions from higher modes to the total response.

7 Conclusions

Two analysis procedures are available for earthquake analysis of concrete gravity dams including dam-water-foundation interaction: (1) response spectrum analysis (RSA) in which the peak response is estimated directly from the earthquake design spectrum; and (2) response history analysis (RHA) of a finite element idealization of the dam monolith. The investigation presented in this report has led to the following conclusions:

1. Analyses of an actual dam to an ensemble of 58 ground motions has demonstrated that the RSA procedure estimates dam response that is close enough to the "exact" response determined by the RHA procedure. Thus, the RSA procedure is satisfactory for the preliminary phase of the design of new dams and in the safety evaluation of existing dams.

2. To enhance the accuracy of this RSA procedure, the possibility of calculating stresses by finite element analysis versus the commonly used beam formulas was investigated, and a correction factor for beam stresses on the downstream face of the dam has been developed.

3. A more complete set of data for the parameters that characterize dam-foundation interaction in the RSA procedure has been developed. Availability of these data should provide sufficient control over the overall damping in the dam-water-foundation system to ensure consistency with damping measured from motions of dams recorded during forced vibration tests and earthquakes.

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NOTATION

The following symbols are used in this report:

$A\left(\tilde{T}_1, \tilde{\zeta}_1\right)$	pseudo-acceleration spectrum ordinate evaluated at natural period $\tilde{T}_1$ and damping ratio $\tilde{\zeta}_1$
A_p	integral of $2gp(\hat{y})/wH$ over depth of the impounded water for $H/H_s = 1$ as listed in Table A.5
a_g	peak ground acceleration
B_1	defined in Equation (2.11)
C	velocity of pressure waves in water
E_f	Young's modulus of elasticity of foundation rock
E_s	Young's modulus of elasticity of dam concrete
F_{st}	$= ½wH^2$, hydrostatic force
$f_1(y)$	equivalent static lateral forces acting on the upstream face of the dam due to the fundamental mode of vibration, as defined in Equation (2.1)
$f_{\text{sc}}(y)$	equivalent static lateral forces acting on the upstream face of the dam due to higher modes of vibration, as defined in Equation (2.10)
g	acceleration due to gravity
H	depth of impounded water
H_s	height of upstream face of dam
L_1	generalized earthquake force coefficient, defined in Equation (2.5)
$\tilde{L}_1$	integral defined in Equation (2.3)
M_1	generalized mass of dam, defined in Equation (2.4)
$\tilde{M}_1$	integral defined in Equation (2.2)
$p\left(y, \tilde{T}_r\right)$	real-valued component of the complex-valued function representing the hydrodynamic pressure on the upstream face due to harmonic acceleration at period $\tilde{T}_r$ in the shape of the fundamental mode
$p_0(y)$	hydrodynamic pressure on a rigid dam with water compressibility neglected
R_f	period lengthening ratio due to dam-foundation interaction
R_r	period lengthening ratio due to dam-water interaction
R_w	$= T_1^r / \tilde{T}_r$
r_1	response due to earthquake forces associated with the fundamental mode of vibration
$r_{\max}$	peak earthquake response of the dam including initial static effects
r_{sc}	response due to earthquake forces associated with the higher modes of vibration
r_{st}	response due to initial static effects
T_1	fundamental vibration period of dam on rigid foundation with empty reservoir given by Equation (3.1)
$\tilde{T}_1$	fundamental resonant period of dam on flexible foundation with impounded water given by Equation (2.8)
T_1^r	$= 4H/C$, fundamental vibration period of impounded water

$\tilde{T}_f$	fundamental resonant period of dam on flexible foundation with empty reservoir given by Equation (2.7)
$\tilde{T}_r$	fundamental resonant period of dam on rigid foundation with impounded water given by Equation (2.6)
t	time
w	unit weight of water
$w_s(y)$	weight of dam per unit height
x	coordinate along the breadth of the dam
y	coordinate along the height of the dam
$\hat{y}$	$= y / H$
α	wave reflection coefficient for reservoir bottom materials
$\tilde{\Gamma}_1$	$= \tilde{L}_1 / \tilde{M}_1$
$\phi_1(y)$	fundamental vibration mode shape of dam at upstream face
η_f	constant hysteretic damping factor for foundation rock
ζ_1	damping ratio of dam on rigid foundation with empty reservoir
$\tilde{\zeta}_1$	damping ratio for dam on flexible foundation with impounded water
$\tilde{\zeta}_f$	added damping due to dam-foundation interaction
$\tilde{\zeta}_r$	added damping due to dam-water interaction
$\sigma_{y,1}$	vertical stress due to earthquake forces associated with the fundamental mode of vibration
$\sigma_{y,d}$	earthquake induced vertical stress
$\sigma_{y,\mathrm{sc}}$	vertical stress due to earthquake forces associated with the higher modes of vibration
σ_d	peak value of maximum principal stress

Appendix A Tables for Standard Values Used in Analysis Procedure

Table A.1 Standard fundamental mode shape $\phi_1(y)$ for concrete gravity dams.

y/H_s	$\phi_1(y)$
1.0	1.000
0.95	.866
0.90	.735
0.85	.619
0.80	.530
0.75	.455
0.70	.389
0.65	.334
0.60	.284
0.55	.240
0.50	.200
0.45	.165
0.40	.135
0.35	.108
0.30	.084
0.25	.065
0.20	.047
0.15	.034
0.10	.021
0.05	.010
0	0

Table A.2(a) **Standard values for R_r and ζ_r, the period lengthening ratio and added damping ratio due to hydrodynamic effects for modulus of elasticity of concrete, E_s = 5 and 4.5 million psi.**

		E_s = 5 million psi		E_s = 4.5 million psi	
H/H_s	α	R_r	ζ_r	R_r	ζ_r
1.0	1.0	1.454	0	1.409	0
	0.90	1.462	.043	1.416	.030
	0.75	1.456	.060	1.412	.051
	0.50	1.355	.067	1.344	.060
	0.25	1.284	.054	1.285	.050
	0	1.261	.038	1.259	.036
0.95	1.0	1.368	0	1.323	0
	0.90	1.376	.044	1.330	.031
	0.75	1.366	.056	1.323	.049
	0.50	1.255	.060	1.256	.053
	0.25	1.208	.045	1.208	.042
	0	1.192	.032	1.191	.030
0.90	1.0	1.289	0	1.247	0
	0.90	1.297	.041	1.253	.029
	0.75	1.284	.050	1.247	.042
	0.50	1.181	.050	1.185	.044
	0.25	1.151	.036	1.152	.033
	0	1.139	.025	1.139	.023
0.85	1.0	1.215	0	1.179	0
	0.90	1.224	.033	1.185	.023
	0.75	1.206	.042	1.177	.034
	0.50	1.129	.039	1.131	.033
	0.25	1.111	.027	1.109	.025
	0	1.100	.019	1.099	.018
0.80	1.0	1.148	0	1.121	0
	0.90	1.156	.024	1.126	.015
	0.75	1.140	.032	1.121	.024
	0.50	1.092	.028	1.092	.024
	0.25	1.078	.019	1.078	.018
	0	1.071	.014	1.071	.013
0.75	1.0	1.092	0	1.078	0
	0.90	1.099	.014	1.080	.008
	0.75	1.089	.021	1.078	.014
	0.50	1.065	.018	1.064	.015
	0.25	1.055	.013	1.055	.012
	0	1.049	.009	1.050	.009

Table A.2(a) – continued.

		E_s = 5 million psi		E_s = 4.5 million psi	
H/H_s	α	R_r	ζ_r	R_r	ζ_r
0.70	1.0	1.055	0	1.048	0
	0.90	1.057	.006	1.050	.003
	0.75	1.055	.011	1.050	.007
	0.50	1.045	.011	1.044	.009
	0.25	1.038	.009	1.037	.008
	0	1.034	.006	1.035	.006
0.65	1.0	1.033	0	1.031	0
	0.90	1.034	.002	1.031	.001
	0.75	1.034	.005	1.031	.003
	0.50	1.030	.006	1.029	.005
	0.25	1.026	.005	1.027	.005
	0	1.024	.004	1.025	.004
0.60	1.0	1.020	0	1.020	0
	0.90	1.020	.001	1.020	.001
	0.75	1.020	.002	1.020	.001
	0.50	1.019	.003	1.018	.003
	0.25	1.017	.003	1.018	.003
	0	1.016	.003	1.016	.002
0.55	1.0	1.013	0	1.012	0
	0.90	1.013	.000	1.012	.000
	0.75	1.013	.001	1.012	.001
	0.50	1.013	.002	1.012	.001
	0.25	1.012	.002	1.012	.002
	0	1.011	.002	1.012	.001
0.50	1.0	1.009	0	1.008	0
	0.90	1.009	.000	1.008	.000
	0.75	1.009	.000	1.008	.000
	0.50	1.008	.001	1.008	.001
	0.25	1.008	.001	1.008	.001
	0	1.008	.001	1.008	.001

Table A.2(b) **Standard values for R_r and ζ_r, the period lengthening ratio and added damping ratio due to hydrodynamic effects for modulus of elasticity of concrete, E_s = 4, 3.5 and 3 million psi.**

		E_s = 4 million psi		E_s = 3.5 million psi		E_s = 3 million psi	
H/H_s	α	R_r	ζ_r	R_r	ζ_r	R_r	ζ_r
1.0	1.0	1.370	0	1.341	0	1.320	0
	0.90	1.374	.021	1.344	.013	1.319	.008
	0.75	1.374	.040	1.341	.029	1.312	.021
	0.50	1.333	.051	1.316	.042	1.289	.035
	0.25	1.285	.045	1.282	.040	1.264	.036
	0	1.259	.034	1.256	.032	1.247	.030
0.95	1.0	1.289	0	1.259	0	1.241	0
	0.90	1.292	.020	1.263	.012	1.240	.007
	0.75	1.289	.038	1.259	.027	1.233	.019
	0.50	1.247	.045	1.238	.036	1.213	.030
	0.25	1.208	.038	1.208	.033	1.194	.030
	0	1.191	.028	1.188	.026	1.181	.025
0.90	1.0	1.214	0	1.191	0	1.176	0
	0.90	1.220	.017	1.193	.010	1.176	.006
	0.75	1.214	.033	1.193	.022	1.171	.015
	0.50	1.179	.037	1.174	.029	1.155	.024
	0.25	1.152	.030	1.152	.026	1.141	.024
	0	1.139	.022	1.136	.020	1.131	.019
0.85	1.0	1.152	0	1.136	0	1.126	0
	0.90	1.157	.013	1.139	.007	1.125	.004
	0.75	1.155	.024	1.136	.016	1.122	.011
	0.50	1.129	.028	1.124	.023	1.111	.017
	0.25	1.109	.022	1.109	.020	1.101	.017
	0	1.099	.017	1.099	.016	1.093	.015
0.80	1.0	1.104	0	1.095	0	1.087	0
	0.90	1.106	.008	1.094	.004	1.087	.003
	0.75	1.106	.016	1.090	.011	1.085	.007
	0.50	1.089	.019	1.080	.016	1.079	.012
	0.25	1.078	.016	1.071	.014	1.071	.012
	0	1.071	.012	1.066	.011	1.066	.011
0.75	1.0	1.070	0	1.063	0	1.059	0
	0.90	1.069	.004	1.063	.003	1.059	.002
	0.75	1.065	.010	1.061	.006	1.058	.004
	0.50	1.056	.013	1.055	.010	1.054	.007
	0.25	1.050	.011	1.050	.010	1.050	.008
	0	1.046	.009	1.046	.008	1.046	.007

Table A.2(b) – continued.

H/H_s	α	E_s = 4 million psi		E_s = 3.5 million psi		E_s = 3 million psi	
		R_r	ζ_r	R_r	ζ_r	R_r	ζ_r
0.70	1.0	1.044	0	1.041	0	1.039	0
	0.90	1.044	.002	1.041	.001	1.039	.001
	0.75	1.042	.005	1.040	.003	1.038	.002
	0.50	1.038	.007	1.037	.006	1.036	.004
	0.25	1.034	.007	1.034	.006	1.034	.005
	0	1.031	.006	1.031	.005	1.031	.005
0.65	1.0	1.028	0	1.026	0	1.025	0
	0.90	1.028	.001	1.026	.001	1.025	.000
	0.75	1.027	.002	1.026	.002	1.025	.001
	0.50	1.025	.004	1.024	.003	1.024	.002
	0.25	1.023	.004	1.022	.004	1.022	.003
	0	1.021	.004	1.021	.003	1.021	.003
0.60	1.0	1.017	0	1.016	0	1.016	0
	0.90	1.017	.000	1.016	.000	1.016	.000
	0.75	1.017	.001	1.016	.001	1.016	.001
	0.50	1.016	.002	1.015	.002	1.015	.001
	0.25	1.015	.002	1.014	.002	1.014	.002
	0	1.013	.002	1.013	.002	1.013	.002
0.55	1.0	1.010	0	1.010	0	1.010	0
	0.90	1.010	.000	1.010	.000	1.010	.000
	0.75	1.010	.001	1.010	.000	1.010	.000
	0.50	1.010	.001	1.010	.001	1.009	.001
	0.25	1.009	.001	1.009	.001	1.009	.001
	0	1.009	.001	1.009	.001	1.009	.001
0.50	1.0	1.006	0	1.006	0	1.006	0
	0.90	1.006	.000	1.006	.000	1.006	.000
	0.75	1.006	.000	1.006	.000	1.006	.000
	0.50	1.006	.001	1.006	.001	1.006	.001
	0.25	1.005	.001	1.005	.001	1.005	.001
	0	1.005	.001	1.005	.001	1.005	.001

Table A.2(c) **Standard values for R_r and ζ_r, the period lengthening ratio and added damping ratio due to hydrodynamic effects for modulus of elasticity of concrete, E_s = 2.5, 2 and 1 million psi.**

		E_s = 2.5 million psi		E_s = 2 million psi		E_s = 1 million psi	
H/H_s	α	R_r	ζ_r	R_r	ζ_r	R_r	ζ_r
1.0	1.0	1.301	0	1.286	0	1.263	0
	0.90	1.301	.005	1.285	.003	1.263	.001
	0.75	1.287	.014	1.284	.009	1.262	.004
	0.50	1.283	.025	1.275	.018	1.260	.008
	0.25	1.264	.030	1.262	.024	1.256	.013
	0	1.247	.027	1.247	.024	1.247	.017
0.95	1.0	1.224	0	1.212	0	1.193	0
	0.90	1.224	.005	1.211	.003	1.193	.001
	0.75	1.221	.012	1.210	.008	1.193	.003
	0.50	1.209	.022	1.203	.015	1.191	.007
	0.25	1.194	.025	1.192	.020	1.187	.011
	0	1.181	.022	1.181	.020	1.181	.014
0.90	1.0	1.164	0	1.154	0	1.140	0
	0.90	1.163	.004	1.154	.002	1.140	.001
	0.75	1.161	.009	1.152	.006	1.140	.002
	0.50	1.152	.017	1.148	.012	1.139	.005
	0.25	1.141	.020	1.140	.016	1.136	.008
	0	1.131	.018	1.131	.016	1.131	.011
0.85	1.0	1.117	0	1.110	0	1.100	0
	0.90	1.116	.003	1.110	.002	1.100	.001
	0.75	1.115	.007	1.109	.004	1.100	.002
	0.50	1.109	.012	1.106	.009	1.100	.004
	0.25	1.101	.014	1.100	.012	1.097	.006
	0	1.093	.013	1.093	.012	1.093	.008
0.80	1.0	1.081	0	1.077	0	1.071	0
	0.90	1.081	.002	1.077	.001	1.071	.000
	0.75	1.080	.004	1.076	.003	1.071	.001
	0.50	1.076	.008	1.074	.006	1.070	.003
	0.25	1.071	.010	1.071	.008	1.069	.005
	0	1.066	.010	1.066	.008	1.066	.006
0.75	1.0	1.055	0	1.053	0	1.049	0
	0.90	1.055	.001	1.053	.001	1.049	.000
	0.75	1.054	.003	1.052	.002	1.049	.001
	0.50	1.053	.005	1.051	.004	1.048	.002
	0.25	1.050	.007	1.049	.005	1.048	.003
	0	1.046	.007	1.046	.006	1.046	.004

Table A.2(c) – continued.

		E_s = 2.5 million psi		E_s = 2 million psi		E_s = 1 million psi	
H/H_s	α	R_r	ζ_r	R_r	ζ_r	R_r	ζ_r
0.70	1.0	1.037	0	1.035	0	1.033	0
	0.90	1.037	.001	1.035	.000	1.033	.000
	0.75	1.037	.002	1.035	.001	1.033	.000
	0.50	1.035	.003	1.034	.002	1.033	.001
	0.25	1.033	.004	1.033	.004	1.032	.002
	0	1.031	.004	1.031	.004	1.031	.003
0.65	1.0	1.024	0	1.023	0	1.022	0
	0.90	1.024	.000	1.023	.000	1.022	.000
	0.75	1.024	.001	1.023	.001	1.022	.000
	0.50	1.023	.002	1.023	.001	1.022	.001
	0.25	1.022	.003	1.022	.002	1.021	.001
	0	1.021	.003	1.021	.003	1.021	.002
0.60	1.0	1.016	0	1.016	0	1.014	0
	0.90	1.016	.000	1.016	.000	1.014	.000
	0.75	1.016	.001	1.016	.001	1.014	.000
	0.50	1.015	.001	1.015	.001	1.014	.000
	0.25	1.014	.002	1.014	.002	1.014	.001
	0	1.013	.002	1.013	.002	1.013	.001
0.55	1.0	1.009	0	1.009	0	1.009	0
	0.90	1.009	.000	1.009	.000	1.009	.000
	0.75	1.009	.000	1.009	.000	1.009	.000
	0.50	1.009	.001	1.009	.000	1.009	.000
	0.25	1.009	.001	1.009	.001	1.009	.000
	0	1.009	.001	1.009	.001	1.009	.001
0.50	1.0	1.006	0	1.006	0	1.005	0
	0.90	1.006	.000	1.006	.000	1.005	.000
	0.75	1.006	.000	1.006	.000	1.005	.000
	0.50	1.006	.000	1.005	.000	1.005	.000
	0.25	1.005	.000	1.005	.000	1.005	.000
	0	1.005	.001	1.005	.000	1.005	.000

Table A.3 **Standard values for R_f and ζ_f, the period lengthening ratio and added damping ratio due to dam-foundation interaction.**

		Added damping ratio, ζ_f									
E_f / E_s	R_f	η_f =.01	η_f =.02	η_f =.03	η_f =.04	η_f =.05	η_f =.06	η_f =.07	η_f =.08	η_f =.09	η_f =.10
5.0	1.044	.011	.011	.011	.012	.012	.013	.013	.013	.014	.014
4.5	1.049	.012	.012	.013	.013	.014	.014	.015	.015	.015	.016
4.0	1.054	.013	.014	.014	.015	.015	.016	.016	.017	.017	.018
3.5	1.061	.016	.016	.017	.017	.018	.018	.019	.019	.020	.020
3.0	1.070	.018	.019	.020	.020	.021	.021	.022	.023	.023	.024
2.5	1.083	.022	.023	.024	.024	.025	.026	.026	.027	.028	.028
2.0	1.102	.028	.029	.030	.030	.031	.032	.033	.034	.035	.035
1.5	1.131	.037	.038	.039	.040	.041	.042	.043	.045	.046	.047
1.4	1.139	.040	.041	.042	.043	.044	.045	.046	.048	.049	.050
1.3	1.149	.043	.044	.045	.046	.047	.049	.050	.051	.052	.053
1.2	1.159	.046	.047	.049	.050	.051	.052	.054	.055	.056	.057
1.1	1.172	.050	.051	.053	.054	.055	.057	.058	.059	.061	.062
1.0	1.187	.054	.056	.057	.059	.060	.062	.063	.065	.066	.067
0.9	1.204	.060	.062	.063	.065	.066	.068	.069	.071	.072	.074
0.8	1.225	.066	.068	.070	.072	.073	.075	.077	.078	.080	.082
0.7	1.252	.075	.076	.078	.080	.082	.084	.086	.087	.089	.091
0.6	1.286	.085	.087	.089	.091	.093	.095	.097	.099	.101	.103
0.5	1.332	.097	.100	.102	.104	.107	.109	.111	.114	.116	.118
0.4	1.396	.115	.117	.120	.123	.125	.128	.130	.133	.136	.138
0.3	1.495	.138	.141	.145	.148	.151	.154	.157	.160	.163	.166
0.2	1.670	.173	.177	.181	.185	.189	.193	.197	.201	.205	.208

Table A.3 – continued.

	Added damping ratio, ζ_f						
E_f / E_s	$\eta_f = 0.12$	$\eta_f = 0.14$	$\eta_f = 0.16$	$\eta_f = 0.18$	$\eta_f = 0.20$	$\eta_f = 0.25$	$\eta_f = 0.50$
5.0	.015	.016	.016	.017	.018	.019	.025
4.5	.017	.017	.018	.019	.020	.021	.027
4.0	.019	.020	.020	.021	.022	.024	.030
3.5	.021	.022	.023	.024	.025	.027	.035
3.0	.025	.026	.027	.028	.029	.032	.040
2.5	.030	.031	.032	.034	.035	.038	.047
2.0	.037	.039	.040	.042	.043	.046	.058
1.5	.049	.051	.052	.054	.056	.060	.075
1.4	.052	.054	.056	.058	.060	.064	.080
1.3	.055	.058	.060	.062	.064	.068	.085
1.2	.060	.062	.064	.066	.068	.073	.091
1.1	.064	.067	.069	.072	.074	.079	.098
1.0	.070	.073	.075	.078	.080	.086	.107
0.9	.077	.080	.082	.085	.088	.094	.117
0.8	.085	.088	.091	.094	.097	.104	.129
0.7	.095	.098	.101	.105	.108	.115	.143
0.6	.107	.111	.114	.118	.121	.130	.162
0.5	.122	.127	.131	.135	.139	.149	.186
0.4	.143	.148	.153	.158	.163	.174	.220
0.3	.172	.179	.185	.191	.196	.211	.269
0.2	.216	.224	.232	.240	.247	.266	.351

Table A.4(a) Standard values for the hydrodynamic pressure function $p(\hat{y})$ for full reservoir, i.e., H/H_s = 1; α = 1.0.

$\hat{y} = y/H$	Value of $gp(\hat{y})/wH$												
	$R_w \le .5$	R_w=.7	R_w=.8	R_w=.85	R_w=.90	R_w=.92	R_w=.93	R_w=.94	R_w=.95	R_w=.96	R_w=.97	R_w=.98	R_w=.99
1.00	0	0	0	0	0	0	0	0	0	0	0	0	0
0.95	.070	.073	.076	.079	.083	.086	.088	.090	.092	.096	.102	.111	.133
0.90	.112	.118	.124	.129	.138	.143	.147	.151	.157	.164	.176	.195	.238
0.85	.127	.135	.144	.152	.164	.172	.178	.184	.193	.204	.221	.249	.313
0.80	.133	.144	.155	.165	.182	.193	.200	.208	.220	.235	.257	.295	.379
0.75	.141	.154	.168	.180	.201	.214	.223	.234	.248	.267	.294	.340	.445
0.70	.145	.161	.178	.192	.216	.232	.242	.255	.272	.294	.327	.382	.506
0.65	.143	.161	.180	.197	.224	.242	.254	.269	.288	.313	.351	.414	.558
0.60	.139	.159	.180	.199	.230	.250	.264	.280	.301	.330	.373	.444	.605
0.55	.137	.159	.183	.203	.237	.260	.274	.293	.316	.348	.395	.473	.651
0.50	.135	.159	.184	.206	.244	.269	.284	.304	.329	.364	.415	.500	.694
0.45	.130	.155	.182	.206	.246	.272	.289	.310	.338	.375	.430	.522	.730
0.40	.124	.151	.179	.204	.247	.275	.293	.315	.345	.384	.442	.540	.762
0.35	.121	.149	.179	.205	.250	.279	.298	.322	.353	.395	.456	.559	.793
0.30	.118	.147	.178	.206	.252	.283	.303	.328	.360	.403	.467	.575	.820
0.25	.113	.143	.175	.204	.252	.284	.304	.330	.363	.408	.475	.587	.840
0.20	.109	.139	.172	.202	.252	.284	.305	.332	.366	.412	.481	.596	.856
0.15	.107	.138	.172	.202	.252	.286	.307	.334	.369	.417	.487	.604	.871
0.10	.106	.137	.172	.202	.253	.287	.309	.337	.372	.420	.491	.611	.881
0.05	.103	.135	.169	.200	.252	.286	.308	.336	.372	.420	.492	.613	.886
0	.100	.133	.168	.198	.251	.285	.307	.335	.371	.420	.492	.613	.886

Table A.4(b) Standard values for the hydrodynamic pressure function $p(\hat{y})$ for full reservoir, i.e., H/H_s = 1; α = 0.90.

$\hat{y} = y / H$	Value of $gp(\hat{y}) / wH$								
	R_w≤.5	R_w=.7	R_w=.8	R_w=.9	R_w=.95	R_w=1.0	R_w=1.05	R_w=1.1	R_w=1.2
1.00	0	0	0	0	0	0	0	0	0
0.95	.070	.073	.076	.082	.088	.089	.069	.064	.062
0.90	.112	.118	.124	.136	.149	.149	.110	.100	.095
0.85	.127	.135	.144	.162	.181	.181	.123	.108	.101
0.80	.133	.144	.155	.179	.204	.205	.127	.107	.098
0.75	.141	.154	.168	.197	.228	.229	.133	.108	.097
0.70	.145	.161	.177	.212	.249	.249	.135	.105	.092
0.65	.143	.161	.179	.219	.261	.262	.130	.096	.081
0.60	.139	.159	.179	.234	.271	.272	.124	.085	.067
0.55	.137	.159	.182	.231	.283	.283	.119	.076	.057
0.50	.135	.159	.183	.236	.293	.292	.114	.067	.046
0.45	.130	.155	.181	.238	.299	.298	.106	.055	.032
0.40	.124	.150	.178	.238	.303	.301	.097	.044	.019
0.35	.121	.148	.177	.241	.309	.307	.091	.035	.009
0.30	.118	.146	.177	.243	.313	.311	.086	.027	.000
0.25	.113	.142	.174	.242	.315	.312	.078	.017	.000
0.20	.109	.139	.171	.241	.316	.312	.071	.008	.000
0.15	.107	.137	.170	.242	.318	.313	.067	.003	.000
0.10	.106	.136	.170	.242	.320	.313	.064	.000	.000
0.05	.103	.134	.167	.241	.318	.311	.059	.000	.000
0	.101	.133	.166	.239	.317	.309	.056	.000	.000

Table A.4(c) **Standard values for the hydrodynamic pressure function $p(\hat{y})$ for full reservoir, i.e., H/H_s = 1; α = 0.75.**

$\hat{y} = y / H$	Value of $gp(\hat{y}) / wH$								
	R_w≤.5	R_w=.7	R_w=.8	R_w=.9	R_w=.95	R_w=1.0	R_w=1.05	R_w=1.1	R_w=1.2
1.00	0	0	0	0	0	0	0	0	0
0.95	.070	.073	.075	.079	.080	.078	.073	.068	.065
0.90	.112	.118	.122	.129	.132	.128	.118	.101	.101
0.85	.127	.133	.140	.151	.154	.150	.134	.121	.110
0.80	.133	.143	.152	.166	.171	.163	.142	.125	.110
0.75	.140	.153	.164	.181	.187	.177	.151	.130	.110
0.70	.145	.159	.173	.193	.200	.188	.157	.131	.108
0.65	.143	.159	.174	.197	.205	.191	.155	.126	.099
0.60	.139	.157	.174	.199	.208	.192	.151	.118	.088
0.55	.137	.157	.175	.203	.213	.195	.150	.113	.079
0.50	.135	.156	.176	.206	.216	.196	.147	.107	.070
0.45	.129	.152	.173	.205	.216	.194	.140	.097	.058
0.40	.123	.147	.170	.203	.214	.191	.134	.088	.045
0.35	.120	.145	.169	.204	.215	.190	.129	.080	.036
0.30	.117	.143	.168	.204	.215	.188	.125	.074	.027
0.25	.112	.139	.164	.201	.212	.184	.118	.065	.016
0.20	.108	.135	.161	.199	.209	.180	.111	.056	.007
0.15	.106	.134	.159	.198	.208	.177	.107	.051	.001
0.10	.104	.133	.158	.197	.207	.175	.103	.046	.000
0.05	.102	.130	.156	.194	.204	.171	.098	.040	.000
0	.100	.128	.154	.192	.201	.167	.093	.036	.000

Table A.4(d) **Standard values for the hydrodynamic pressure function $p(\hat{y})$ for full reservoir, i.e., H/H_s = 1; α = 0.50.**

$\hat{y} = y / H$	Value of $gp(\hat{y}) / wH$								
	R_w≤.5	R_w=.7	R_w=.8	R_w=.9	R_w=.95	R_w=1.0	R_w=1.05	R_w=1.1	R_w=1.2
1.00	0	0	0	0	0	0	0	0	0
0.95	.071	.072	.073	.074	.074	.073	.072	.070	.068
0.90	.112	.116	.118	.119	.119	.118	.116	.113	.108
0.85	.125	.132	.135	.136	.135	.134	.130	.127	.120
0.80	.132	.139	.143	.146	.145	.143	.138	.133	.123
0.75	.139	.148	.153	.156	.155	.152	.146	.139	.127
0.70	.144	.154	.160	.163	.162	.158	.151	.143	.128
0.65	.141	.152	.159	.163	.161	.156	.148	.138	.122
0.60	.137	.149	.157	.162	.160	.153	.143	.132	.113
0.55	.135	.148	.156	.161	.158	.151	.141	.128	.107
0.50	.133	.147	.155	.159	.156	.148	.137	.123	.099
0.45	.127	.142	.150	.154	.151	.142	.129	.115	.088
0.40	.121	.136	.145	.149	.145	.136	.122	.106	.077
0.35	.117	.133	.143	.146	.142	.131	.116	.099	.069
0.30	.114	.131	.140	.143	.137	.126	.110	.092	.060
0.25	.109	.126	.135	.137	.131	.119	.102	.083	.050
0.20	.104	.121	.130	.132	.125	.112	.094	.074	.040
0.15	.102	.119	.127	.128	.121	.108	.089	.068	.033
0.10	.100	.117	.125	.125	.118	.104	.083	.062	.026
0.05	.098	.114	.121	.121	.113	.098	.077	.055	.018
0	.096	.111	.119	.117	.108	.093	.072	.049	.012

Table A.4(e) **Standard values for the hydrodynamic pressure function $p(\hat{y})$ for full reservoir, i.e., H/H_s = 1; α = 0.25.**

$\hat{y} = y / H$	Value of $gp(\hat{y}) / wH$								
	R_w≤.5	R_w=.7	R_w=.8	R_w=.9	R_w=.95	R_w=1.0	R_w=1.05	R_w=1.1	R_w=1.2
1.00	0	0	0	0	0	0	0	0	0
0.95	.069	.070	.071	.071	.071	.071	.070	.070	.070
0.90	.111	.113	.114	.114	.114	.114	.113	.113	.111
0.85	.124	.127	.128	.129	.129	.128	.127	.127	.125
0.80	.130	.133	.134	.135	.135	.134	.133	.132	.129
0.75	.137	.141	.142	.143	.142	.141	.140	.138	.135
0.70	.141	.145	.147	.147	.146	.145	.143	.141	.137
0.65	.137	.142	.144	.144	.143	.142	.140	.137	.131
0.60	.133	.138	.140	.139	.138	.136	.134	.131	.124
0.55	.131	.136	.137	.136	.135	.133	.130	.126	.118
0.50	.128	.133	.134	.133	.131	.128	.125	.121	.112
0.45	.121	.126	.127	.126	.124	.120	.116	.112	.101
0.40	.115	.120	.120	.118	.115	.112	.107	.102	.091
0.35	.111	.116	.116	.113	.110	.106	.100	.095	.082
0.30	.107	.111	.111	.107	.104	.099	.093	.087	.074
0.25	.101	.105	.104	.100	.096	.091	.084	.077	.063
0.20	.096	.099	.098	.093	.088	.082	.076	.068	.052
0.15	.094	.096	.094	.088	.083	.076	.069	.061	.044
0.10	.092	.096	.090	.083	.078	.071	.063	.054	.037
0.05	.088	.088	.085	.077	.071	.064	.055	.046	.028
0	.086	.085	.081	.072	.065	.057	.048	.039	.020

Table A.4(f) **Standard values for the hydrodynamic pressure function $p(\hat{y})$ for full reservoir, i.e., H/H_s = 1; α = 0.**

$\hat{y} = y / H$	Value of $gp(\hat{y}) / wH$								
	R_w≤.5	R_w=.7	R_w=.8	R_w=.9	R_w=.95	R_w=1.0	R_w=1.05	R_w=1.1	R_w=1.2
1.00	0	0	0	0	0	0	0	0	0
0.95	.069	.069	.069	.069	.069	.069	.070	.070	.070
0.90	.109	.110	.110	.111	.111	.111	.112	.112	.112
0.85	.122	.123	.124	.125	.125	.125	.126	.126	.126
0.80	.127	.128	.128	.129	.129	.129	.130	.130	.130
0.75	.133	.134	.134	.135	.135	.135	.136	.136	.136
0.70	.135	.136	.137	.138	.138	.138	.139	.139	.139
0.65	.132	.133	.133	.133	.133	.133	.134	.134	.134
0.60	.127	.127	.127	.127	.127	.127	.127	.127	.127
0.55	.123	.123	.123	.123	.123	.123	.122	.122	.121
0.50	.120	.119	.118	.118	.118	.117	.116	.116	.115
0.45	.113	.111	.110	.109	.109	.108	.107	.106	.105
0.40	.105	.103	.102	.100	.099	.098	.097	.096	.094
0.35	.101	.098	.096	.094	.092	.091	.090	.088	.085
0.30	.096	.092	.090	.087	.085	.084	.082	.080	.076
0.25	.090	.085	.082	.078	.076	.074	.072	.069	.065
0.20	.084	.078	.074	.070	.067	.065	.062	.059	.053
0.15	.080	.073	.068	.064	.061	.058	.055	.051	.045
0.10	.077	.069	.064	.058	.054	.051	.048	.044	.036
0.05	.073	.063	.057	.050	.046	.043	.039	.035	.026
0	.070	.058	.052	.044	.040	.036	.031	.027	.017

Table A.5(a) **Standard values for A_p, the hydrodynamic force coefficient in $\tilde{L}_1$; α = 1.0.**

R_w	Value of A_p for α=1
0.99	1.242
0.98	.893
0.97	.739
0.96	.647
0.95	.585
0.94	.539
0.93	.503
0.92	.474
0.90	.431
0.85	.364
0.80	.324
0.70	.279
≤ 0.50	.237

Table A.5(b) **Standard values for A_p, the hydrodynamic force coefficient in $\tilde{L}_1$; α = 0.90, 0.75, 0.50, 0.25 and 0.**

	Value of A_p				
R_w	α=0.90	α=0.75	α=0.50	α=0.25	α=0
1.20	.071	.111	.159	.178	.181
1.10	.110	.177	.204	.197	.186
1.05	.194	.249	.229	.205	.189
1.00	.515	.340	.252	.213	.191
0.95	.518	.378	.267	.219	.193
0.90	.417	.361	.274	.224	.195
0.80	.322	.309	.269	.229	.198
0.70	.278	.274	.256	.228	.201
≤ 0.50	.237	.236	.231	.222	.206

Table A.6 **Standard values for the hydrodynamic pressure function $p_0(\hat{y})$.**

$\hat{y} = y / H$	gp_0 / wH
1.0	0
0.95	.137
0.90	.224
0.85	.301
0.80	.362
0.75	.418
0.70	.465
0.65	.509
0.60	.546
0.55	.580
0.50	.610
0.45	.637
0.40	.659
0.35	.680
0.30	.696
0.25	.711
0.20	.722
0.15	.731
0.10	.737
0.05	.741
0	.742

Appendix B Probabilistic Seismic Hazard Analysis for Pine Flat Dam Site

Summarized in this appendix is the probabilistic seismic hazard analysis (PSHA) performed for the Pine Flat Dam site to obtain the ensemble of ground motions used in the response analysis presented in Chapter 6.

B.1 TARGET SPECTRUM

Figure B.1 shows two Conditional Mean Spectra (CMS) for the Pine Flat Dam site computed by the procedure in Baker [2011] at the 1% in 100 years hazard level for the intensity measures $A(T_1)$ and $A(\tilde{T}_1)$, where $T_1 \approx 0.3$ sec and $\tilde{T}_1 \approx 0.5$ sec are the fundamental vibration periods of the dam alone on a rigid foundation and the dam with impounded water on flexible foundation, respectively. These values cover the range of periods for the four analysis cases listed in Table 6.1.

It was decided to evaluate the accuracy of the RSA procedure using the same ensemble of ground motions for all the four analysis cases considered; thus ground motions were selected and scaled for a single target spectrum. Because the two CMS corresponding to the periods T_1 and $\tilde{T}_1$ are very similar, the target spectrum is, for convenience, taken as the geometric mean of the two CMS, shown in Figure B.1. Although more rigorous procedures exist for computing CMS for an intensity measure that averages spectral acceleration values over a range of periods [Baker and Cornell 2006], the target spectrum selected is considered satisfactory for the limited objective of comparing the RSA and RHA procedures.

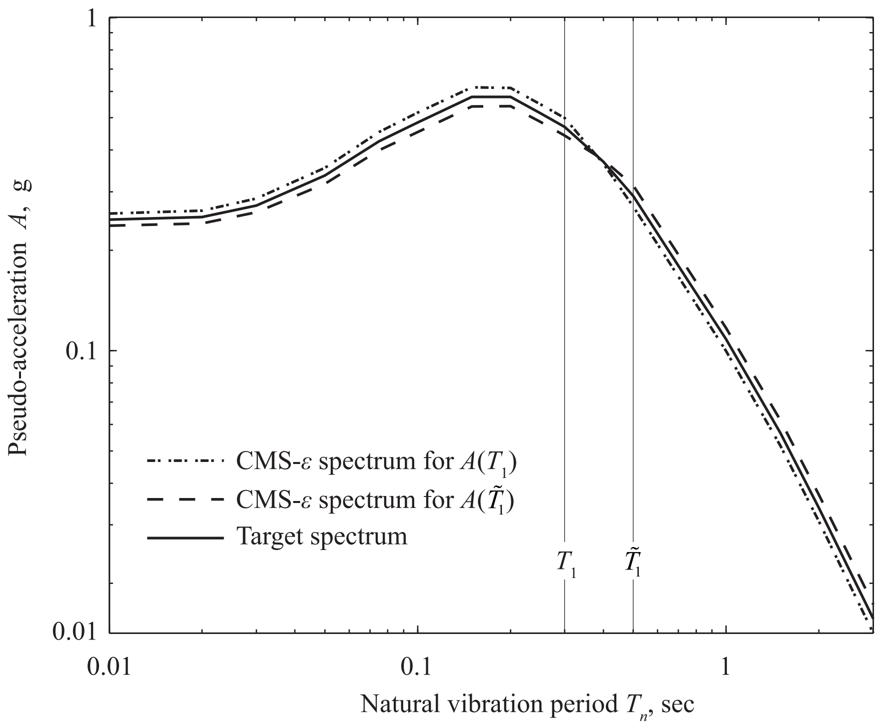

Figure B.1 **CMS-** ε **spectra for intensity measures** $A(T_1)$ **and** $A(\tilde{T}_1)$ **at the 1% in 100 years hazard level. Also plotted is the target spectrum; damping,** ζ **= 5%.**

B.2 SELECTION AND SCALING OF GROUND MOTIONS

The 29 acceleration records listed in Table B.1, each with two orthogonal horizontal components, were selected from the PEER Ground Motion Database [PEER Ground Motion 2010] according to the following criteria:

- Fault distance, R = 0–50 km
- Magnitude, M_w = 5–7.5
- Shear wave velocity, $V_{s,30}$ > 183 m/sec (corresponding to minimum NEHRP soil class D, stiff soil).

The range of M_w and R were selected to be consistent with the deaggregation of the seismic hazard at the Pine Flat Dam site [USGS Deaggregation 2008] where it was clear that the dominant events at the site for the main periods of interest were close distance earthquakes in magnitude range M_w = 5 - 7.5. The range of $V_{s,30}$ was chosen to discard ground motions recorded on very soft soils, which are not representative for the rock site at Pine Flat Dam.

The selected records were amplitude-scaled by scaling each ground motion to minimize the mean square difference between the response spectrum for the individual ground motion and the target spectrum over the period range of interest. A detailed description of this scaling procedure can be found in PEER Ground Motion [2010].

Figure B.2 presents the response spectra for the scaled ground motions, the target spectrum, and the median (computed as the geometric mean) of the 58 response spectra.

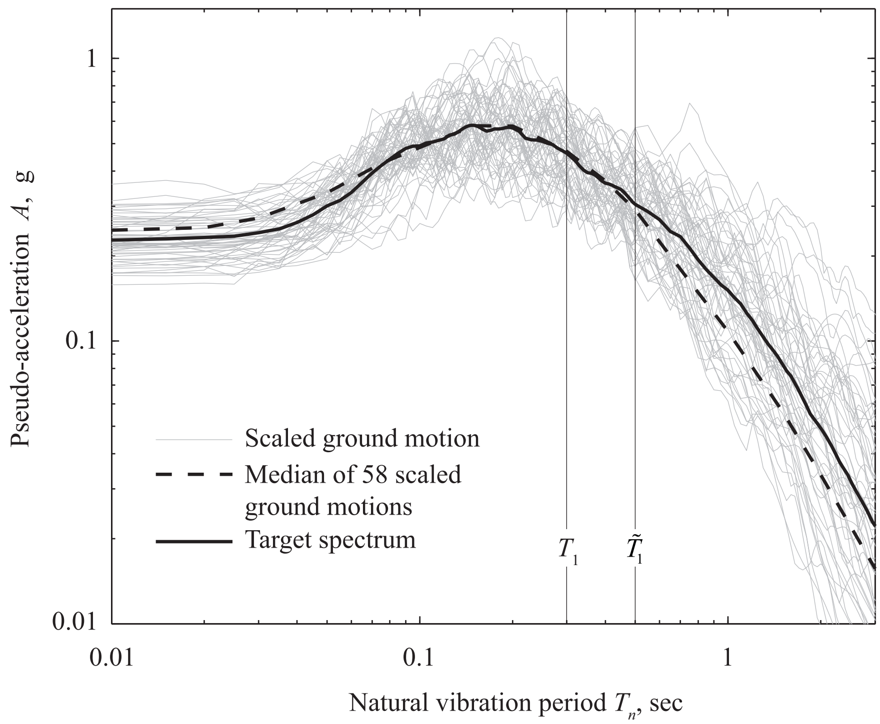

Figure B.2 **Response spectra for 58 scaled ground motion records, their median spectrum, and the target spectrum; damping, ζ = 5%.**

Table B.1 List of earthquake records. PGA values are for the scaled fault-normal and fault-parallel components of the ground motions.

#	Year	Event	Station	M_w	R, in km.	PGA, in g FN comp.	PGA, in g FP comp.
1	1966	Parkfield	Cholame Shandon Array	6.19	17.6	0.232	0.246
2	1971	San Fernando	LA - Hollywood Stor FF	6.61	22.8	0.180	0.229
3	1971	San Fernando	Lake Hughes 4	6.61	25.1	0.256	0.319
4	1979	Imperial Valley	Victoria	6.53	31.9	0.179	0.306
5	1980	Mammoth Lakes	Mammoth Lakes H.S.	6.06	4.7	0.179	0.271
6	1980	Irpinia, Italy	Auletta	6.90	9.5	0.198	0.211
7	1980	Irpinia, Italy	Rionero In Vulture	6.90	30.1	0.226	0.210
8	1983	Mammoth Lakes	Convict Creek	5.31	7.1	0.191	0.313
9	1983	Coalinga 05	Oil Fields Fire Station FF	5.77	11.1	0.292	0.243
10	1984	Morgan Hill	Gilroy Array #2	6.19	13.7	0.278	0.228
11	1986	N. Palm Springs	San Jacinto - Valley Cemetary	6.06	31.0	0.253	0.219
12	1986	N. Palm Springs	Sunnymead	6.06	37.9	0.236	0.227
13	1986	Chalfant Valley	Benton	6.19	21.9	0.251	0.214
14	1987	Whittier Narrows	Glendale - Las Palmas	5.99	22.8	0.312	0.189
15	1987	Whittier Narrows	Glendora - N. Oakbank	5.99	22.1	0.282	0.205
16	1987	Whittier Narrows	LA - Century City CC North	5.99	29.9	0.188	0.275
17	1987	Whittier Narrows	Pomona - 4th&Locust FF	5.99	29.6	0.262	0.224
18	1987	Whittier Narrows	LA - Hollywood Stor FF	5.27	24.8	0.200	0.278
19	1992	Landers	Mission Creek Fault	7.28	27.0	0.223	0.231
20	1994	Northridge	Burbank - Howard Rd	6.69	16.9	0.134	0.171
21	1994	Northridge	LA - Centinela St	6.69	28.3	0.198	0.300
22	1994	Northridge	LA - Obregon Park	6.69	37.4	0.370	0.197
23	1994	Northridge	LA - Wonderland Ave	6.69	20.3	0.243	0.183
24	1994	Northridge	Santa Monica City Hall	6.69	26.4	0.216	0.324
25	1999	Hector Mine	Twentynine Palms	7.13	42.1	0.215	0.220
26	1999	Chi-Chi, Taiwan	TCU079	6.20	8.5	0.260	0.200
27	1999	Chi-Chi, Taiwan	TCU054	6.20	49.5	0.210	0.266
28	1999	Chi-Chi, Taiwan	TCU075	6.30	26.3	0.300	0.163
29	1999	Chi-Chi, Taiwan	TCU120	6.30	32.5	0.243	0.221

Appendix C Detailed Calculations for Pine Flat Dam

This appendix presents detailed calculations of the equivalent lateral earthquake forces and earthquake induced stresses in Pine Flat Dam that were presented in Chapter 6. The appendix consists of two parts: (1) a summary of the computational steps required in the RSA procedure; and (2) a brief summary of the procedure for obtaining stresses in the RHA procedure using a newer version of the computer program EAGD-84.

C.1 RESPONSE SPECTRUM ANALYSIS PROCEDURE

The dam is analyzed for the four analysis cases listed in Table C.2. For each case the equivalent static lateral forces are computed by implementing the step-by-step procedure presented in Chapter 4, and stresses are computed using the methods described in the subsequent sections. All computations are performed for a unit width of the dam monolith.

Simplified Block Model of Dam Monolith

The simplified model of the tallest, non-overflow cross-section of Pine Flat Dam is shown in Figure C.1. The cross-section is divided into 10 blocks of equal height of 40 ft, the properties of each of the blocks are presented in Table C.1. The total weight of the dam in the simplified block model is 9486 kips, and the modal parameters L_1 and M_1 are computed by replacing the integrals in Equations (2.4) and (2.5) by their respective summations over all the blocks, which yields L_1 = (1390 kips) / g and M_1 = (500 kips) / g.

Table C.1 Properties of each block in the simplified model.

Block	Weight, w, kips	Elevation of centroid, ft.	ϕ_1 at centroid	$w\phi_1$, kips	$w\phi_1^2$, kips
1	202.8	379.9	0.865	175.4	151.8
2	267.3	338.5	0.612	163.7	100.2
3	417.7	298.6	0.450	188.1	84.7
4	610.8	258.9	0.331	202.3	67.0
5	816.7	219.2	0.238	194.6	46.4
6	1022.5	179.3	0.164	167.7	27.5
7	1228.3	139.4	0.107	131.8	14.2
8	1434.2	99.5	0.065	92.6	6.0
9	1640.0	59.6	0.034	55.3	1.9
10	1845.9	19.6	0.010	18.1	0.2
Total	9486			1390	500

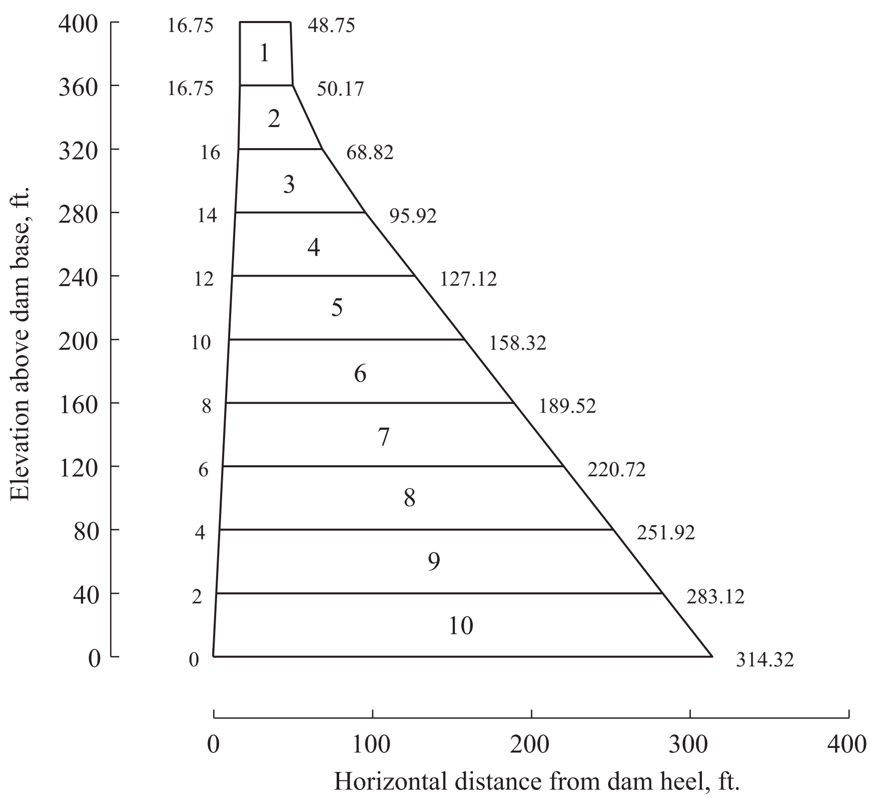

Figure C.1 Coordinates of simplified block model.

Computation of Equivalent Static Lateral Forces

The equivalent static lateral forces associated with the fundamental mode, f_1, and higher modes, f_{sc}, are computed by implementing the step-by-step procedure described in Chapter 4. The details of the computational steps are summarized in this section.

1. For $E_s = 3.25$ million psi and $H_s = 400$ ft., T_1 is computed from Equation (3.1) as $T_1 = (1.4)(400)/\sqrt{3.25 \cdot 10^6} = 0.311$ sec.

2. For $E_s = 3.25$ million psi, $\alpha = 0.75$ and $H/H_s = 381/400 = 0.95$, Table A.2(b) gives $R_r = 1.246$ (linearly interpolated between values for $E_s = 3.0$ million psi and $E_s = 3.5$ million psi), so $\tilde{T}_r = (1.240)(0.311) = 0.387$ sec.

3. The fundamental vibration period for the impounded water is $T_1^r = 4H/C = 4(381)/4720 = 0.323$ sec, Equation (3.2) then gives $R_w = 0.323/0.387 = 0.83$.

4. For $E_f / E_s = 1$, Table A.3 gives $R_f = 1.187$, leading to $\tilde{T}_1 = (1.187)(0.311) = 0.369$ sec for Case 3, and $\tilde{T}_1 = (1.187)(0.387) = 0.459$ sec for Case 4.

5. For Cases 2 and 4, Table A.2(b) gives $\zeta_r = 0.023$ for $E_s = 3.25$ million psi (interpolated), $\alpha = 0.75$, and $H/H_s = 0.95$. For Cases 3 and 4, $\zeta_f = 0.059$ from Table A.3 for $E_f / E_s = 1$ and $\eta_f = 0.04$. With $\zeta_1 = 0.02$, Equation (2.9) then gives: $\tilde{\zeta}_1 = 0.02/1.246 + 0.023 = 0.039$ for Case 2; $\tilde{\zeta}_1 = 0.02/(1.187)^3 + 0.059 = 0.071$ for Case 3; and $\tilde{\zeta}_1 = 0.02/[(1.24)(1.187)^3] + 0.023 + 0.059 = 0.092$ for Case 4.

6. The values of $gp(y)$ presented in Table C.3 at eleven equally spaced levels were obtained from Table A.4(c) for $R_w = 0.83$ (by linearly interpolating between the data for the two closest values for which data are available, $R_w = 0.80$ and $R_w = 0.90$) and $\alpha = 0.75$, and multiplied by (0.0624)(381)(.95)^2 = 21.6 k/ft.

7. Evaluating Equation (2.4) in discrete form gives $M_1 = (500\,\text{kip})/g$. From Equation (3.3), $\tilde{M}_1 = (1.246)^2(1/g)(500) = (776\,\text{kip})/g$.

8. Evaluating Equation (2.5) in discrete form gives $L_1 = (1390\,\text{kip})/g$. From Table A.5(b), $A_p = 0.327$ for $\alpha = 0.75$ and $R_w = 0.83$ (interpolated). Equation (3.4) then gives $\tilde{L}_1 = 1390/g + (1/g)(4529)(0.95)^2(0.327) = (2732\,\text{kip})/g$. Consequently, for Cases 1 and 3, $\tilde{\Gamma}_1 = L_1/M_1 = 1390/500 = 2.78$, and for Cases 2 and 4, $\tilde{\Gamma}_1 = \tilde{L}_1/\tilde{M}_1 = 2732/776 = 3.52$.

9. For each of the four cases listed in Table C.2, Equation (2.1) was evaluated at eleven equally spaced intervals along the height of the dam, including the top and bottom, by substituting values for $\tilde{\Gamma}_1 = \tilde{L}_1 / \tilde{M}_1$ and $gp(y)$ computed in the preceding steps; computing the weight of the dam per unit height $w_s(y)$ from the monolith dimensions shown in Figure C.1 and the unit weight of concrete; and substituting $\phi_1(y)$ from Table A.1 and the pseudo-acceleration ordinate $A(\tilde{T}_1, \tilde{\zeta}_1)$ from the median pseudo-acceleration response spectrum in Figure 6.2 corresponding to the $\tilde{T}_1$ and $\tilde{\zeta}_1$ computed in Steps 4 and

5. The resulting equivalent static lateral forces $f_1(y)$ are presented in Table C.4 for each case, with intermediate values shown in Table C.3.

10. The vertical stresses $\sigma_{y,1}$ due to the response of the dam in its fundamental mode are computed by a static stress analysis of the dam subjected to the equivalent static lateral forces $f_1(y)$ from Step 9 applied to the upstream face of the dam. A summary of the static stress analysis is presented in the next subsection.

11. For each of the four cases, Equation (2.10) was evaluated at eleven equally spaced intervals along the height of the dam, including the top and bottom, by substituting numerical values for the quantities computed in the preceding steps; obtaining $gp_0(y)$ from Table A.6; using Equation (2.11) to compute $B_1 = (0.20)(4529/g)(0.95)^2 = (817.5\text{kip})/g$, which yields $B_1/M_1 = 817.5/500 = 1.64$; and substituting $a_g = 0.232$ g. The resulting equivalent static lateral forces $f_{\text{sc}}(y)$ are presented in Table C.4 for each case, with intermediate values shown in Table C.3.

12. The vertical stresses $\sigma_{y,\text{sc}}$ due to the response of the dam in all higher modes are computed by a static stress analysis of the dam subjected to the equivalent static lateral forces $f_{\text{sc}}(y)$ from Step 11 applied to the upstream face of the dam. A summary of the static stress analysis is presented in the next subsection.

13. Computation of the earthquake induced vertical stresses $\sigma_{y,d}$ is done by combining the response quantities $\sigma_{y,1}$ and $\sigma_{y,\text{sc}}$ computed in Steps 10 and 12 by the SRSS combination rule; this is described in a later subsection.

Table C.2 Analysis cases, fundamental mode properties and pseudo-acceleration values.

Analysis Case	Foundation	Water	$\tilde{\Gamma}_1 = \tilde{L}_1/\tilde{M}_1$	$\tilde{T}_1$, in sec	$\tilde{\zeta}_1$, in percent	$A(\tilde{T}_1, \tilde{\zeta}_1)$, in g
1	Rigid	Empty	2.78	0.311	2.0	0.606
2	Rigid	Full	3.52	0.387	3.9	0.409
3	Flexible	Empty	2.78	0.369	7.1	0.347
4	Flexible	Full	3.52	0.459	9.2	0.274

Table C.3 **Intermediate values for calculation of equivalent static lateral forces.**

y, ft.	w_s, k/ft.	ϕ_1	$w_s\phi_1$, k/ft.	$w_s[1-(L_1/M_1)\phi_1]$, k/ft.	gp, k/ft.	gp_0 k/ft.	$gp_0-(B_1/M_1)w_s\phi_1$, k/ft.
400	4.96	1.000	4.96	-8.83	0	0	-8.16
360	5.18	0.735	3.81	-5.41	1.75	3.47	-2.79
320	8.19	0.530	4.34	-3.88	3.16	7.45	0.31
280	12.7	0.389	4.94	-1.04	3.73	10.3	2.15
240	17.8	0.284	5.07	3.75	3.94	12.5	4.12
200	23.0	0.200	4.60	10.20	3.99	14.1	6.59
160	28.1	0.135	3.80	17.57	3.94	15.6	9.21
120	33.3	0.084	2.80	25.51	3.87	16.4	11.8
80	38.4	0.047	1.81	33.41	3.76	17.1	14.1
40	43.6	0.021	0.92	41.03	3.69	17.5	16.0
0	48.7	0	0	48.72	3.60	17.6	17.6

Table C.4 **Equivalent static lateral forces in kips/ft on Pine Flat Dam.**

y, ft.	Case 1		Case 2		Case 3		Case 4	
	f_1	f_{sc}	f_1	f_{sc}	f_1	f_{sc}	f_1	f_{sc}
400	8.31	- 2.05	7.02	- 3.94	4.74	- 2.05	4.78	- 3.94
360	6.38	- 1.25	7.86	- 1.90	3.64	- 1.25	5.36	- 1.90
320	7.27	- 0.90	10.6	- 0.83	4.15	- 0.90	7.24	- 0.83
280	8.28	- 0.24	12.3	0.26	4.72	- 0.24	8.36	0.26
240	8.49	0.87	12.8	1.83	4.85	0.87	8.69	1.83
200	7.71	2.37	12.2	3.90	4.40	2.37	8.28	3.90
160	6.37	4.08	11.0	6.21	3.63	4.08	7.47	6.21
120	4.69	5.92	9.44	8.66	2.67	5.92	6.43	8.66
80	3.03	7.75	7.88	11.0	1.73	7.75	5.37	11.0
40	1.53	9.52	6.52	13.2	0.88	9.52	4.44	13.2
0	0.00	11.3	5.10	15.4	0.00	11.3	3.47	15.4

Computation of Vertical Stresses

The vertical stresses $\sigma_{y,1}$ and $\sigma_{y,\mathrm{sc}}$ due to each set of equivalent static lateral forces $f_1(y)$ and $f_{\mathrm{sc}}(y)$, respectively, are computed by static stress analysis of the dam monolith by two different methods: (1) stresses at both faces of the dam are computed by elementary formulas for stresses in beams; and (2) stresses are computed by a finite element analysis.

Results are presented in this section for analysis case 4 only, as the computational steps are identical for all the four analysis cases.

Beam Theory

The inertia forces associated with the mass—given by the first term of Equations (2.1) and (2.10)—are applied at the centroid of each of the 10 blocks shown in Figure C.1, and the forces associated with hydrodynamic pressure—given by the second term of the same equations—are applied as a linearly distributed load on the upstream face of each block. The resulting bending moments in the dam monolith are computed at each level from the equilibrium equations, and the normal bending stresses at two faces are computed by elementary beam theory as $\sigma_y = M / S$, where M and S are the bending moment and section modulus, respectively, at the horizontal section considered; these stresses act in the vertical direction. The procedure is implemented in a newly developed computer program similar to the computer program SIMPL described in Appendix D of Fenves and Chopra [1986]. The vertical stresses computed at the two faces of Pine Flat Dam are listed in Table C.5 for analysis case 4.

The stresses with their algebraic signs shown in Table C.5 will occur on the upstream face of the dam when the earthquake forces act in the downstream direction, and on the downstream face of the dam when the earthquake forces act in the upstream direction. The stresses on the sloping part of the downstream face are subsequently multiplied by the correction factor of 0.75 developed in Section 4.3.

Table C.5 Vertical stresses $\sigma_{y,1}$ and $\sigma_{y,\mathrm{sc}}$ for analysis case 4 computed by elementary beam theory.

		Fundamental mode		Higher modes	
y, ft.	Section modulus, $S = 1/6b^2$, ft^3	Bending moment, k-ft.	Vertical stress at faces, psi	Bending moment, k-ft.	Vertical stress at faces, psi
400	171	0	0	0	0
360	186	3,479	130	-2,579	-96
320	465	15,577	233	-8,632	-129
280	1,118	39,103	243	-16,060	-100
240	2,208	75,854	239	-23,020	-72
200	3,665	126,35	239	-26,978	-51
160	5,490	190,037	240	-24,673	-31
120	7,683	265,64	240	-12,398	-11
80	10,242	351,517	238	13,675	9
40	13,170	446,139	235	57,289	30
0	16,464	547,841	231	122,028	51

Finite Element Method

The forces $f_1(y)$ and $f_{sc}(y)$ are applied as linearly distributed forces to the upstream face of the finite element discretization of the dam shown in Figure C.2. Static analysis of the finite element model leads to stresses at the centroid of each element, and a stress recovery procedure is applied in order to obtain stresses at the nodal points.

The resulting vertical stresses $\sigma_{y,1}$ and $\sigma_{y,\text{sc}}$, at the nodal points on the two faces of the dam due to earthquake forces applied in the downstream direction are listed in Table C.6 for analysis case 4. Applying the forces in the upstream direction reverses the algebraic signs of the stresses; numerical values remain unchanged.

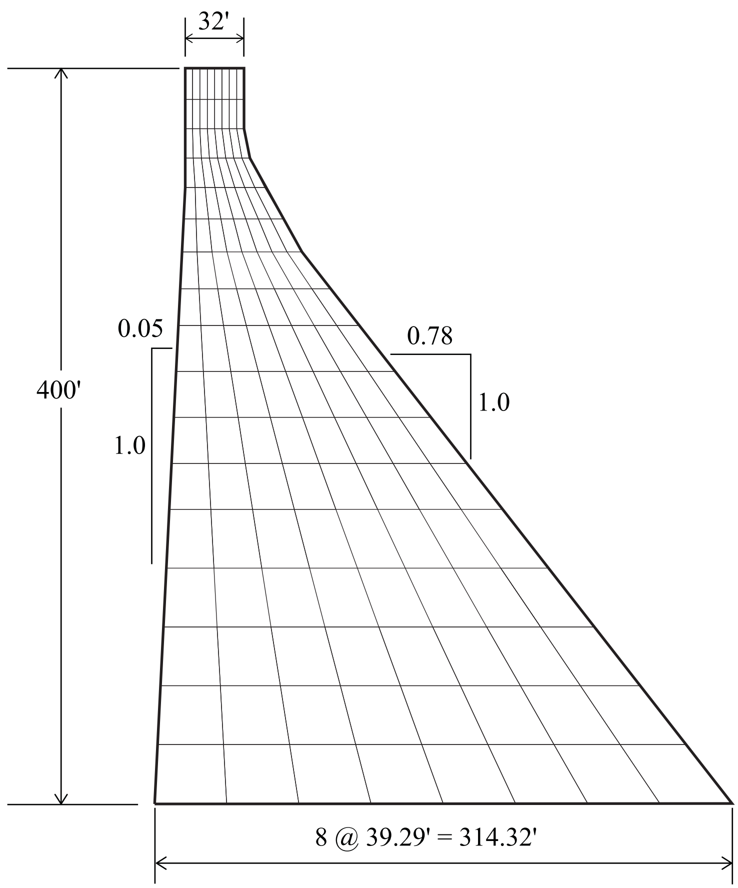

Figure C.2 **Finite element model of Pine Flat Dam used for stress computations in the RSA procedure; mesh consists of 136 quadrilateral four-node elements. The same mesh is used in the RHA procedure.**

Table C.6 **Vertical stresses $\sigma_{y,1}$ and $\sigma_{y,\mathrm{sc}}$, in psi, for analysis case 4 computed by finite element analysis.**

	Fundamental mode		Higher modes	
Height, *y*, ft.	Vertical stress at u/s face	Vertical stress at d/s face	Vertical stress at u/s face	Vertical stress at d/s face
400	12	-9	-9	7
383	34	-34	-24	25
367	92	-108	-61	71
351	160	-183	-98	110
335	209	-207	-118	111
318	232	-214	-119	100
300	240	-216	-110	88
280	243	-200	-98	69
260	241	-190	-86	54
235	239	-190	-73	42
210	237	-190	-62	30
185	237	-190	-52	18
160	238	-185	-43	4
128	241	-176	-32	-9
96	249	-161	-19	-19
64	264	-140	3	-26
32	290	-118	44	-27
0	306	-107	71	-27

Response Combination

The vertical stress at a location due to earthquake excitation is computed by combining $\sigma_{y,1}$ and $\sigma_{y,\mathrm{sc}}$ by the SRSS formula:

$$\sigma_{y,d} = \pm\sqrt{\sigma_{y,1}^2 + \sigma_{y,\mathrm{sc}}^2} \tag{C.1}$$

Because the direction of the applied earthquake forces is reversible, these stresses can be either positive (tensile stresses) or negative (compressive stresses).

The earthquake induced vertical stresses for Pine Flat Dam computed by beam theory and the finite element method are summarized in Tables C.7 and C.8 for analysis case 4; stresses computed by beam theory on the sloping part of the downstream face have been modified by the correction factor of 0.75. These results are also presented in Section 6.3.2.

Table C.7 **Vertical stresses $\sigma_{y,d}$, in psi, for analysis case 4 computed by beam theory.**

Height, *y*, ft.	Vertical stress at u/s face	Vertical stress at d/s face
400	0	0
360	162	162
320	266	200
280	263	197
240	250	187
200	245	184
160	242	182
120	240	180
80	239	179
40	237	179
0	237	178

Table C.8 **Vertical stresses $\sigma_{y,d}$, in psi, for analysis case 4 computed by finite element analysis.**

Height, *y*, ft.	Vertical stress at u/s face	Vertical stress at d/s face
400	15	12
383	42	42
367	110	130
351	188	213
335	240	234
318	261	236
300	264	234
280	262	212
260	256	197
235	250	195
210	245	192
185	242	190
160	242	185
128	244	176
96	250	162
64	264	143
32	294	121
0	314	110

Principal Stresses: Beam Theory

At the upstream and downstream faces of the dam, principal stresses due to each of the force distributions f_1 and f_{sc} can be determined by a simple transformation of the corresponding vertical stresses determined by beam theory. If the upstream face of the dam is nearly vertical and the effects of tail-water are negligible, this transformation can be written as [Fenves and Chopra 1986: Appendix C]

$$\sigma_1 = \sigma_{y,1} \sec^2 \theta \tag{C.2a}$$

$$\sigma_{sc} = \sigma_{y,sc} \sec^2 \theta \tag{C.2b}$$

where θ is the angle of the face with respect to the vertical. Under these restricted conditions the principal stresses are directly proportional to the vertical stresses, and hence also to the modal coordinate, therefore modal combination rules are applicable.

The maximum principal stresses on the two faces of the dam computed by combining σ_1 and σ_{sc} using the SRSS formula are shown in Table C.9, where the vertical stresses entering the Equation (C.2) are computed by beam theory. These values are also presented in Section 6.4.2, where they are compared to the results obtained by the RHA procedure.

Table C.9 Maximum principal stresses σ_d, in psi, for analysis case 4 computed by beam theory.

Height, y, ft	Maximum principal stress at u/s face	Maximum principal stress at d/s face
400	0	0
360	162	121
320	266	243
280	263	287
240	250	301
200	245	295
160	243	292
120	241	290
80	239	288
40	238	286
0	237	286

C.2 RESPONSE HISTORY ANALYSIS PROCEDURE

A set of pre- and post-processor scripts were developed to facilitate response history analyses for the 58 ground motions in the computer program EAGD-84 [Fenves and Chopra 1984c], this program provides stresses as a function of time for every element in the finite element model (mesh shown in Figure C.2). From the stress response histories the peak values of the maximum principal stress over the duration of each ground motion are determined, and the median value at every nodal point on the two faces is computed as the geometric mean of the stress values due to the 58 ground motions.

Such results are presented in Figure C.3 for the four analysis cases; the median results are also presented in Section 6.4.2 where they are compared with stresses computed by the RSA procedure.

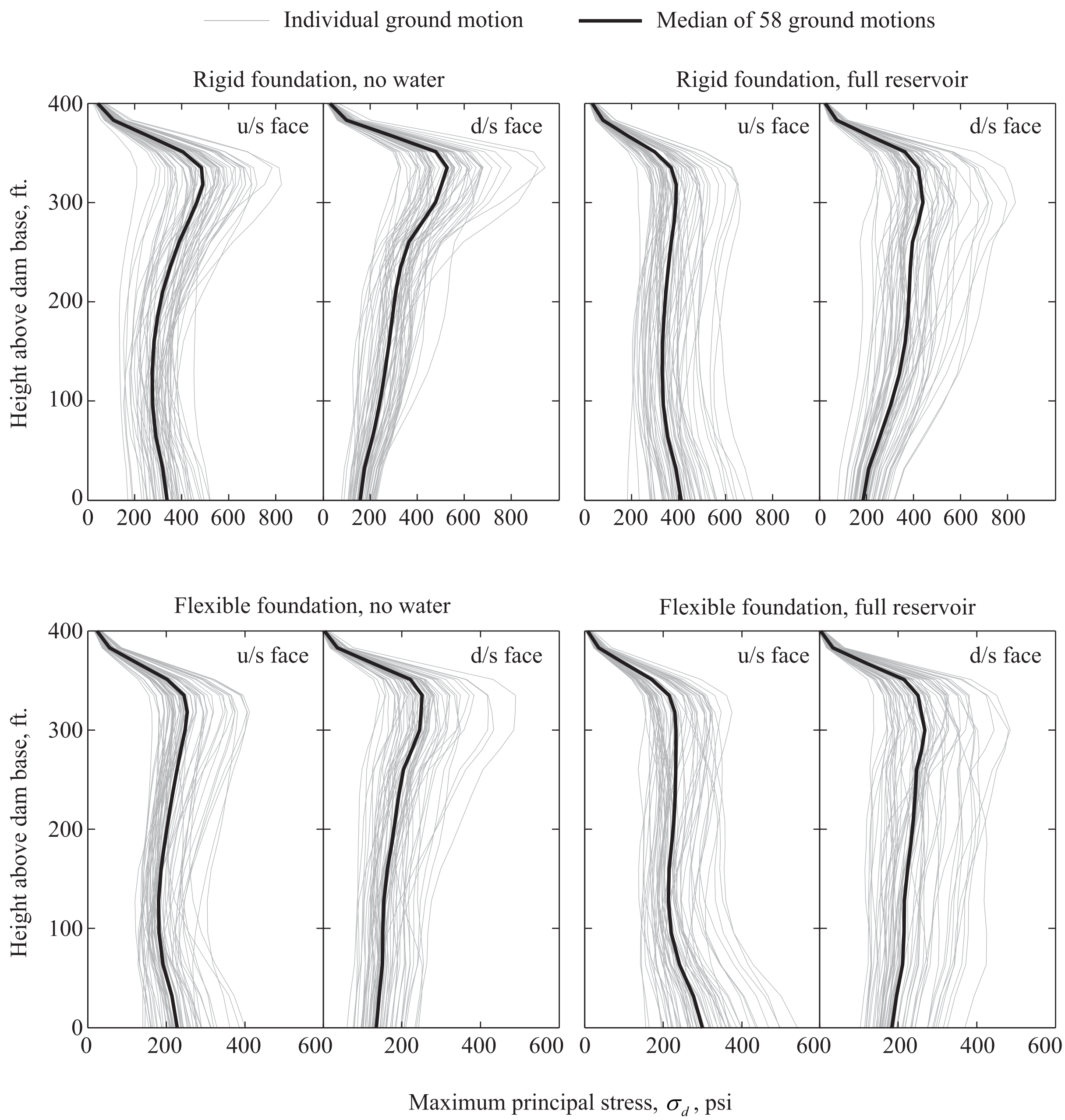

Figure C.3 **Peak maximum principal stresses, σ_d, at the two faces of Pine Flat Dam due to each of the 58 ground motions computed by RHA. Also plotted are the median values.**

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APPENDIX E –SIMPLIFIED EARTHQUAKE ANALYSIS OF GATED SPILLWAY MONOLITHS OF CONCRETE GRAVITY DAMS

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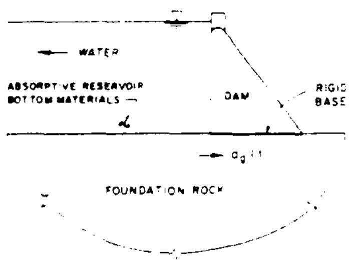

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SIMPLIFIED EARTHQUAKE ANALYSIS OF GATED SPILLWAY MONOLITHS OF CONCRETE GRAVITY DAMS

by

Anil K. Chopra, Hanchen Tan

University of California at Berkeley
Berkeley, California 94720

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19 ABSTRACT (Continue on reverse if necessary and identify by block number)

The simplified procedure for earthquake analysis of concrete gravity dams developed earlier for nonoverflow monoliths is extended in this report to gated spillway monoliths. Standard data are presented for the vibration properties of such monoliths and for all parameters that are required in the analysis procedure. The use of the simplified analysis procedure and of a computer program that facilitates implementation of the procedure is illustrated by examples.

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18. SUBJECT TERMS (Continued).

Concrete
Dams (gravity)
Design
Dynamics
Earthquakes
Earthquake-resistant structures
Foundation interaction
Hydrodynamic pressure
Spillway

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PREFACE

This report presents a simplified procedure for earthquake analysis of concrete gravity dams for gated spillway monoliths, an extension of an earlier report on nonoverflow monoliths. Standard data are presented for the vibration properties of such monoliths and for all parameters that are required in the analysis procedure. The use of the simplified analysis procedure and of a computer program that facilitates implementation of the procedure is illustrated by examples.

This report was prepared by Dr. Anil K. Chopra, Professor of Civil Engineering, and Hanchen Tan, graduate student, University of California at Berkeley. Funds for this report were provided to the US Army Engineer Waterways Experiment Station (WES) by the Civil Works Research and Development Program of the Office, Chief of Engineers (OCE), under the Structural Engineering Research Program, Contract No. DAAG29-81-D-0100, Delivery Order No. 1855 with Battelle Laboratories, Research Triangle Park, NC.

Vincent P. Chiarito and Dr. Robert L. Hall of the Structural Mechanics Division (SMD), Structures Laboratory (SL), WES, and Lucien G. Guthrie, OCE, Engineering Division, provided input for this work. Dr. Hall prepared the preliminary computer analyses (Reference 6) requested by the authors in order to develop the system idealization used in this report for gated spillway monoliths.

Although this report could have been written as an addendum to the work on nonoverflow monoliths, for the convenience of the user it is organized to be self-contained, but at the expense of extensive duplication with References 2 and 3, which resulted from the work of Dr. Gregory L. Fenves. The investigation was conducted at WES under the general supervision of Messrs. Bryant Mather, Chief, SL, James T. Ballard, Assistant Chief, SL, and Dr. Jimmy P. Balsara, Chief, SMD, SL.

COL Dwayne G. Lee, EN, is the Commander and Director of WES. Dr. Robert W. Whalin is Technical Director.

CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
feet	0.3048	metres
inches	25.4	millimetres
kips (force)	4.448222	kilonewtons
kips (force) per square inch	6.894757	megapascals
miles (US statue)	1.609	kilometres
pounds (force)	4.448222	newtons
pounds (force) per foot	14.593904	newtons per metre
pounds (force) per square inch	0.006894757	megapascals

SIMPLIFIED EARTHQUAKE ANALYSIS OF GATED SPILLWAY MONOLITHS OF CONCRETE GRAVITY DAMS

INTRODUCTION

A simplified analysis procedure was presented in 1978 for elastic design and safety evaluation of concrete dams [1]. In this procedure the lateral earthquake forces associated with the fundamental vibration mode of nonoverflow monoliths are estimated directly from the earthquake design spectrum, considering the effects of dam-water interaction and water compressibility. More recently [2,3], the procedure has been extended to also consider the absorption of hydrodynamic pressure waves in the reservoir bottom sediments and in the underlying foundation rock. Also included in the newer simplified procedure is a "static correction" method to consider the response contributions of the higher vibration modes, and a rule for combining the modal responses.

The objective of this report is to extend the above-mentioned simplified analysis procedure for nonoverflow monoliths to gated spillway monoliths. Utilizing the analytical development underlying the procedure [4,5], this report is concerned with implementation of the procedure. Standard data for the vibration properties of gated spillway monoliths and the quantities that depend on them are presented to minimize the computations. The use of the simplified analysis procedure and a computer program for static stress analysis is illustrated by examples.

SYSTEM IDEALIZATION

A gated spillway monolith of a concrete gravity dam is shown in Fig. 1, including a pier, gate, bridge, and foot bucket. Based on results of finite element analyses, it was concluded that: (1) the fundamental vibration period and mode shape of a spillway monolith are not influenced significantly by the bridge, gate, or foot bucket [6], but the effects of the pier may not be negligible; and (2) an equivalent two-dimensional system of unit thickness along the dam axis, with the mass and elastic modulus of the monolith kept at their actual values, but those of the pier reduced by the ratio of monolith thickness to pier thickness, is satisfactory for computing the fundamental vibration period and mode shape of the system [6].

The equivalent two-dimensional system representing a gated spillway monolith is assumed to be supported on a viscoelastic half-plane and impounding a reservoir of water, possibly with alluvium and sediments at the bottom (Fig. 2). Although the equivalent single-degree-of-freedom (SDF) system representation is valid for dams of any cross-section, the upstream face of the dam was assumed to be vertical [4,5] only for the purpose of evaluating the hydrodynamic terms in the governing equations. The standard data presented in this report are also based on this assumption, which is reasonable for actual concrete gravity dams because the upstream face is vertical or almost vertical for most of the height, and the hydrodynamic pressure on the dam face is insensitive to small departures of the face slope from vertical, especially if these departures are near the base of the dam, which is usually the case. The dynamic effects of the tail water are neglected because it is usually too shallow to influence dam response.

EQUIVALENT LATERAL FORCES FOR FUNDAMENTAL VIBRATION MODE

Considering only the fundamental mode of vibration of the dam, the maximum effects of the horizontal earthquake ground motion can be represented by equivalent lateral forces acting on the upstream face of the spillway monolith and pier [2,3,5]:

$$f_1(y) = \frac{\tilde{L}_1}{\tilde{M}_1} \frac{S_a(\tilde{T}_1, \tilde{\xi}_1)}{g} \left[w_s(y)\, \phi(y) + g p(y, \tilde{T}_r) \right] \tag{1}$$

in which the y-coordinate is measured from the base of the dam along its height and $w_s(y)$ is the weight per unit height of the equivalent two-dimensional system of unit thickness along the dam axis. As mentioned in the preceding section, $w_s(y)$ is equal to the actual weight of the dam monolith per unit thickness, but for the pier it is the weight of the pier per unit thickness divided by the ratio of the monolith thickness to the pier thickness. In Eq. 1,

$$\tilde{M}_1 = M_1 + \int_0^H p(y, \tilde{T}_r)\phi(y)dy \tag{2a}$$

$$M_1 = \frac{1}{g}\int_0^{H_s} w_s(y)\phi^2(y)dy \tag{2b}$$

$$\tilde{L}_1 = L_1 + \int_0^H p(y, \tilde{T}_r)dy \tag{3a}$$

$$L_1 = \frac{1}{g}\int_0^{H_s} w_s(y)\phi(y)dy \tag{3b}$$

M_1 is the generalized mass and L_1 is the generalized earthquake force coefficient; $\phi(y)$ is the horizontal component of displacement at the upstream face of the dam in the fundamental vibration mode shape of the dam supported on rigid foundation rock with empty reservoir; $p(y, T_r) \equiv Re[\overline{p}_1(y, \tilde{T}_r)]$ where $\overline{p}_1$ is the complex-valued function representing the hydrodynamic pressure on the upstream face due to harmonic acceleration of period $\tilde{T}_r$ (defined later) in the fundamental vibration mode; H is the depth of the impounded water; H_s is the height of the dam; g is the acceleration due to gravity; and $S_a(\tilde{T}_1, \xi_1)$ is the pseudo-acceleration ordinate of the earthquake design spectrum evaluated at the vibration period $\tilde{T}_1$ and damping ratio ξ_1 of the equivalent SDF system representing the dam-water-foundation rock system. Equation 1 is an extension of Eq. 9 in Ref. 1 to include the effects of dam-foundation rock interaction and reservoir bottom materials on the lateral forces.

The natural vibration period of the equivalent SDF system representing the fundamental mode response of the dam on rigid foundation rock with impounded water is [4]:

$$\tilde{T}_r = R_r T_1 \tag{4a}$$

in which T_1 is the fundamental vibration period of the dam on rigid foundation rock with empty reservoir. Because of the frequency-dependent, added hydrodynamic mass arising from dam-water interaction, the factor $R_r > 1$. It depends on the properties of the dam, the depth of the water, and the absorptiveness of the reservoir bottom materials. The natural vibration period of the equivalent SDF system representing the fundamental mode response of the dam on flexible foundation rock with emptry reservoir is [4]:

$$\tilde{T}_f = R_f T_1 \tag{4b}$$

Because of the frequency-dependent, added foundation-rock flexibility arising from dam-foundation rock interaction, the factor $R_f > 1$. It depends on the properties of the dam and foundation rock.

The natural vibration period of the equivalent SDF system representing the fundamental mode response of the dam on flexible foundation rock with impounded water is approximately given by [5]:

$$\tilde{T}_1 = R_r R_f T_1 \tag{4c}$$

The damping ratio of this equivalent SDF system is [5]:

$$\tilde{\xi}_1 = \frac{1}{R_r}\frac{1}{(R_f)^3}\xi_1 + \xi_r + \xi_f \tag{5}$$

in which ξ_1 is the damping ratio of the dam on rigid foundation rock with emptry reservoir; ξ_r represents the added damping due to dam-water interaction and reservoir bottom absorption; and ξ_f represents the added radiation and material damping due to dam-foundation rock interaction. Considering that R_r and $R_f > 1$, Eq. 5 shows that dam-water interaction and dam-foundation rock interaction reduce the effectiveness of structural damping. However, usually, this reduction is more than compensated for by the added damping due to reservoir bottom absorption and due to dam-foundation rock interaction, which leads to an increase in the overall damping of the dam.

The quantities R_r, R_f, ξ_r, ξ_f, $p(y, \tilde{T}_r)$, $\tilde{L}_1$, and $\tilde{M}_1$ which are required to evaluate the equivalent lateral forces, Eq. 1, contain all the modifications of the vibration properties of the equivalent SDF system and of the generalized earthquake force coefficient necessary to account for the effecs of dam-water interaction, reservoir bottom absorption, and dam-foundation rock interaction. Even after the considerable simplification necessary to arrive at Eq. 1, its evaluation is still too complicated for practical applications because the aforementioned quantities are complicated fucntions of the hydrodynamic and foundation-rock flexibility terms [5]. Fortunately, as will be seen in a later section, the computation of lateral forces can be considerably simplified by recognizing that the cross-sectional geometry of concrete gravity dams does not vary widely.

STANDARD PROPERTIES FOR FUNDAMENTAL MODE RESPONSE

Direct evaluation of Eq. 1 would require complicated computation of several quantities: $p(y, \tilde{T}_r)$ from an infinite series expression; the period lengthening ratios R_r and R_f due to dam-water and dam-foundation rock interactions by iterative solution of equations involving frequency-dependent terms; damping ratios ξ_f and ξ_r from expressions involving complicated foundation-rock flexibility and hydrodynamic terms; the integrals in Eqs. 2a and 3a; and the fundamental vibration period and mode shape of the dam [4,5]. The required computations would be excessive for purposes of preliminary design of dams. Recognizing that the cross-section geometry of nonoverflow monoliths of concrete gravity dams does not vary widely, standard values for the vibration properties of these monoliths and all quantities that depend on them and enter into Eq. 1 were developed in Refs. 2 and 3. A single set of standard data was sufficient because the standard shape chosen for nonoverflow monoliths was assumed to be appropriate for dams of all heights; i.e. the standard shape multiplied by a height-scaling factor provides the cross-section for a dam of particular height.

However, a single standard shape did not seem appropriate for spillway monoliths because, according to Corps of Engineers staff, the pier height is usually about the same (approximately 60 ft) for a wide range of dam heights. Because a pier of these dimensions would influence the vibration properties of shorter monoliths to a greater degree than it would affect taller monoliths, it was decided to generate two sets of "standard" data, one appropriate for lower dams, defined for purposes of this report as $H_s < 300$ ft; the other for higher dams, defined herein as $H_s \geq 300$ ft. Because the slopes of the downstream and upstream faces of the monolith are usually steeper in higher dams compared to lower dams, cross-sectional geometry appropriate to each of the two cases was selected (Appendix A) in generating the data presented later.

Vibration Properties of the Dam

Computed by the finite element method, the fundamental vibration period, in seconds, of two "standard" cross-sections for spillway monoliths of concrete gravity dams on rigid foundation rock with empty reservoir is (Appendix A):

$$T_1 = 1.2 \frac{H_s}{\sqrt{E_s}} \quad \text{if } H_s \geq 300 \text{ ft} \tag{6a}$$

$$T_1 = 1.25 \frac{H_s}{\sqrt{E_s}} \quad \text{if } H_s < 300 \text{ ft} \tag{6b}$$

in which H_s is the total (monolith plus pier) height of the dam, in feet; and E_s is the Young's modulus of elasticity of concrete, in pounds per square inch. The fundamental vibration mode shape $\phi(y)$ of the two standard cross-sections is shown in Fig. 3 and Table 1.

Modification of Period and Damping: Dam-Water Interaction

Dam-water interaction and reservoir bottom absorption modify the natural vibration period (Eq. 4a) and the damping ratio (Eq. 5) of the equivalent SDF system representing the fundamental vibration mode response of the dam. For a fixed cross-section, the period lengthening ratio R_r and added damping ξ_r depend on several parameters, the more significant of which are: Young's modulus E_s of the dam concrete, ratio H/H_s of water depth to dam height, and wave reflection coefficient α. This coefficient, α, is the ratio of the amplitude of the reflected hydrodynamic pressure wave to the amplitude of a vertically propagating pressure wave incident on the reservoir bottom [4,7,8,9]; $\alpha = 1$ indicates that pressure waves are completed reflected, and smaller values of α indicate increasingly absorptive materials.

The results of many analyses of the two "standard" spillway cross-sections, using the procedures developed in Ref. 4 and modified in Ref. 2 for dams with larger elastic modulus E_s, are summarized in Figs. 4 and 5 and Table 2 for higher dams ($H_s \geq 300$ ft), and in Figs. 6 and 7 and Table 3 for lower dams ($H_s < 300$ ft). The period lengthening ratio R_r and added damping ξ_r are presented as a function of H/H_s for $E_s = 5.0$, 4.5, 4.0, 3.5, 3.0, 2.5, and 2.0 million psi and $\alpha = 1.00$, 0.90, 0.75, 0.50, 0.25, and 0. Whereas the dependence of R_r and ξ_r on E_s, H/H_s and α, and the underlying mechanics of dam-water interaction and reservoir bottom absorption are discussed elsewhere in detail [2,4,5], it is useful to note that R_r increases and ξ_r generally, but not always, increases with increasing water depth, absorptiveness of reservoir bottom materials, and concrete modulus. The effects of dam-water interaction and reservoir bottom absorption may be neglected, and the dam analyzed as if there is no impounded water, if the reservoir depth is small, $H/H_s < 0.5$; in particular, $R_r \approx 1$ and $\xi_r \approx 0$.

Modification of Period and Damping: Dam-Foundation Rock Interaction

Dam-foundation rock interaction modifies the natural vibration period (Eq. 4b) and added damping ratio (Eq. 5) of the equivalent SDF system representing the fundamental vibration mode response of the dam. For a fixed dam cross-section, the period lengthening ratio R_f and the added damping ξ_f due to dam-foundation rock interaction depend on several parameters, the more significant of which are: moduli ratio E_f/E_s, where E_s and E_f are the Young's moduli of the dam concrete and foundation rock, respectively; and the constant hysteretic damping factor η_f for the foundation rock.

The results of many analyses of the two "standard" dam cross-sections, using the procedures developed in Ref. 4, are summarized in Figs. 8-9 and Table 4 for higher dams ($H_s \geq 300$ ft) and in Figs. 10-11 and Table 5 for lower dams ($H_s < 300$ ft). The period lengthening ratio R_f and added damping ξ_f are presented for many values of E_f/E_s between 0.2 and 5.0, and $\eta_f = 0.01$, 0.10, 0.25, and 0.50. Whereas the dependence of R_f and ξ_f on E_f/E_s and η_f, and the underlying mechanics of dam-foundation rock interaction are discussed elsewhere in detail [4,5], it is useful to note that the period ratio R_f and added damping ξ_f increase with decreasing E_f/E_s--which, for a fixed value of E_s, implies an increasingly flexible foundation rock--and increasing hysteretic damping factor η_f. The foundation rock may be considered rigid in the simplified analysis if $E_f/E_s > 4$ because then the effects of dam-foundation rock interaction are negligible.

Hydrodynamic Pressure

In order to provide a convenient means for determining $p(y, \tilde{T}_r)$ in Eqs. 1, 2a and 3a, a nondimensional form of this function, $gp(\hat{y})/wH$, where $\hat{y} = y/H$, and w = the unit weight of water, has been computed from the equations presented in Ref. 4 for $\alpha = 1.0$, 0.90, 0.75, 0.5, 0.25, and 0, and the necessary range of values of

$$R_w = \frac{T_1^r}{\tilde{T}_r} \tag{7}$$

in which the fundamental vibration period of the impounded water $T_1^r = 4H/C$, where C is the velocity of pressure waves in water. The results presented in Fig. 12 and Table 6 are for full reservoir, $H/H_s = 1$. The function $gp(\hat{y})/wH$ for any other value of H/H_s is approximately equal to $(H/H_s)^2$ times the function for $H/H_s = 1$ [1].

Generalized Mass and Earthquake Force Coefficient

The generalized mass $\tilde{M}_1$ (Eq. 2a) of the equivalent SDF system representing the dam, including hydrodynamic effects, can be conveniently cmputed from [4]:

$$\tilde{M}_1 = (R_r)^2 M_1 \tag{8a}$$

in which M_1 is given by Eq. 2b. In order to provide a convenient means to compute the generalized earthquake force coefficient $\tilde{L}_1$, Eq. 3a is expressed as:

$$\tilde{L}_1 = L_1 + \frac{1}{g} F_{st} \left(\frac{H}{H_s} \right)^2 A_p \tag{8b}$$

where $F_{st} = wH^2/2$ is the total hydrodynamic force on the dam, and A_p is the integral of the function $2gp(\hat{y})/wH$ over the depth of the impounded water, for $H/H_s = 1$. The hydrodynamic force coefficient A_p is tabulated in Table 7 for a range of values for the period ratio R_w and the wave reflection coefficient α.

EQUIVALENT LATERAL FORCES FOR HIGHER VIBRATION MODES

Because the earthquake response of short vibration period structures, such as concrete gravity dams, is primarily due to the fundamental mode of vibration, the response contributions of the higher vibration modes have, so far, been neglected in the simplified analysis procedure presented in the preceding sections. However, the height-wise mass distribution of concrete gravity dams is such that the effective mass [10] in the fundamental vibration mode is small, e.g. it is 35 percent of the total mass for the standard nonoverflow section [2]. Thus, the contributions of the higher vibration modes to the earthquake forces may not be negligible, and a simple method to consider them is summarized in this section.

This simple method utilizes three concepts. Firstly, because the periods of the higher vibration modes of concrete gravity dams are very short, the higher vibration modes respond to earthquake ground motion with little dynamic amplification in essentially a static manner, leading to the "static correction" concept [11,12]. Secondly, just as in the case of multistory buildings [13], soil-structure interaction effects may be neglected in computing the contributions of the higher vibration modes to the earthquake response of dams. Thirdly, the effects of dam-water interaction and water compressibility may be neglected in computing the higher mode responses [2]. The maximum earthquake effects associated with the higher vibration modes can then be represented by the equivalent lateral forces [2]:

$$f_{sc}(y) = \frac{a_g}{g}\left\{ w_s(y)\left[1 - \frac{L_1}{M_1}\phi(y)\right] + \left[g\,p_0(y) - \frac{B_1}{M_1} w_s(y)\phi(y)\right]\right\} \tag{9}$$

In Eq. 9, a_g is the maximum ground acceleration, $p_0(y)$ is a real-valued, frequency-independent function for hydrodynamic pressure on a rigid dam undergoing unit acceleration, with water compressibility neglected, both assumptions being consistent with the "static correction" concept; and B_1 provides a measure of the portion of $p_0(y)$ that acts in the fundamental vibration mode. Standard values for $p_0(y)$ are presented graphically in Fig. 13 and Table 8. Using the fundamental mode vibration properties of the two "standard" spillway cross-sections, it can be shown that:

$$B_1 = 0.25\frac{F_{st}}{g}\left(\frac{H}{H_s}\right)^2 \tag{10}$$

for both cross-sections, where F_{st} is the total hydrostatic force on the dam. The shape of only the fundamental vibration mode enters into Eq. 9 and the higher mode shapes are not required, thus simplifying the analysis considerably.

RESPONSE COMBINATION

Dynamic Response

As shown in the preceding two sections, the maximum effects of earthquake ground motion in the fundamental vibration mode of the dam have been represented by equivalent lateral forces $f_1(y)$ and those due to all the higher modes by $f_{sc}(y)$. Static analysis of the dam for these two sets of forces provide the values r_1 and r_{sc} for any response quantity r, e.g. the shear force or bending moment at any horizontal section, or the shear or bending stresses at any point. Because the maximum responses r_1 and r_{sc} do not occur at the same time during the earthquake, they should be combined to obtain an estimate of the dynamic response r_d according to the well known modal combination rule: square-root-of-the-sum-of-squares (SRSS) of modal maxima leading to

$$r_d = \sqrt{(r_1)^2 + (r_{sc})^2} \tag{11}$$

Because the natural frequencies of lateral vibration of a concrete dam are well separated, it is not necessary to include the correlation of modal responses in Eq. 11. In Ref. 2, the SRSS combination rule is shown to be preferable to the sum-of-absolute-values (ABSUM) which may provide an overly conservative result.

The SRSS and ABSUM combination rules are applicable to the computation of any response quantity that is proportional to the generalized modal coordinate responses. Thus, these combination rules are generally inappropriate to determine the principal stresses. However, as shown in Ref. 2, the principal stresses at the faces of a dam monolith may be determined by the SRSS method if the upstream face is nearly vertical and the effects of tail water at the downstream face are small.

Total Response

In order to obtain the total value of any response quantity r, the SRSS estimate of dynamic response r_d should be combined with the static effects r_{st}. The latter may be determined by standard analysis procedures to compute the initial stresses in a dam prior to the earthquake, including effects of the self-weight of the dam, hydrostatic pressures, and temperature changes. In order to recognize that the direction of lateral earthquake forces is reversible, combinations of static and dynamic stresses should allow for the worst case, leading to the maximum value of total response:

$$r_{\max} = r_{st} \pm \sqrt{(r_1)^2 + (r_{sc})^2} \tag{12}$$

This combination of static and dynamic responses is appropriate if r_{st}, r_1, and r_{sc} are oriented similarly. Such is the case for the shearing force or bending moment at any horizontal section, for the shear and bending stresses at any point, but generally not for principal stresses, except under the restricted conditions mentioned above.

SIMPLIFIED ANALYSIS PROCEDURE

The maximum effects of an earthquake on a concrete gravity dam are represented by equivalent lateral forces in the simplified analysis procedure. The lateral forces associated with the fundamental vibration mode are computed to include the effects of dam-water interaction, water compressibility, reservoir bottom absorption, and dam-foundation rock interaction. The response contributions of the higher vibration modes are computed under the assumption that the dynamic amplification of the modes is negligible, the interaction effects between the dam, impounded water, and foundation rock are not significant, and that the effects of water compressibility can be neglected. These approximations provide a practical method for including the most important factors that affect the earthquake response of concrete gravity dams.

Selection of System Parameters

The simplified analysis procedure requires only a few parameters to describe the dam-water-foundation rock system: E_s, ξ_1, H_s, E_f, η_f, H, and α. The complete data necessary to implement this procedure are presented as both figures and tables in this report.

The Young's modulus of elasticity E_s for the dam concrete should be based on the design strength of the concrete or suitable test data, if available. The value of E_s may be modified to recognize the strain rates representative of those the concrete may experience during earthquake motions of the dam [1]. In using the figures and tables mentioned earlier to conservatively include dam-water interaction effects in the computation of earthquake forces (Eq. 1), the E_s value should be rounded down to the nearest value for which data are available: $E_s = 2.0$, 2.5, 3.0, 3.5, 4.0, 4.5 or 5.0 million psi. Forced vibration tests on dams indicate that the viscous damping ratio ξ_1 for concrete dams is in the range of 1 to 3 percent. However, for the large motions and high stresses expected in a dam during intense earthquakes, $\xi_1 = 5$ percent is recommended. The height H_s of the dam is measured from the base and includes the height of the spillway monolith and of the pier.

The Young's modulus of elasticity E_f and constant hysteretic damping coefficient η_f of the foundation rock should be determined from a site investigation and appropriate tests. To be conservative, the value of η_f should be rounded down to the nearest value for which data are available: $\eta_f = 0.01$, 0.10, 0.25, or 0.50, and the value of E_f/E_s should be rounded up to the nearest value for which data are available. In the absence of information on damping properties of the foundation rock, a value of $\eta_f = 0.10$ is recommended.

The depth H of the impounded water is measured from the free surface to the reservoir bottom. It is not necessary for the reservoir bottom and dam base to be at the same elevation. The standard values for unit height of water and velocity of pressures waves in water are w = 62.4 pcf and $C = 4720$ fps, respectively.

It may be impractical to reliably determine the wave reflection coefficient α because the reservoir bottom materials may consist of highly variable layers of exposed bedrock, alluvium, silt, and other sediments, and appropriate site investigation techniques have not been developed. Until such techniques become available, α should be selected to give conservative estimates of the earthquake response, which is appropriate at the preliminary design stage. The wave reflection coefficient is defined as [4,7,8,9]:

$$\alpha = \frac{1 - qC}{1 + qC} \tag{13}$$

where $q = \rho/\rho_r C_r$, $C_r = \sqrt{E_r/\rho_r}$, E_r is the Young's modulus of elasticity of the reservoir bottom materials, and ρ_r is their density, C is the velocity of sound in water, and ρ is the density of water. For rigid reservoir bottom, $C_r = \infty$ and $q = 0$, resulting in $\alpha = 1$. In order to obtain a conservative value of α, the value of q may be based on the properties of the impounded water and only the underlying rock, thus neglecting the additional wave absorptiveness due to the overlying sediments. The estimated value of α should be rounded up to the nearest value for which the figures and tables are presented: α = 1.0, 0.90, 0.75, 0.50, 0.25, and 0.00.

Design Earthquake Spectrum

The horizontal earthquake ground motion is specified by a pseudo-acceleration response spectrum in the simplified analysis procedure. This should be a smooth response spectrum--without the irregularities inherent in response spectra of individual ground motions--representative of the intensity and frequency characteristics of the design earthquakes which should be established after a thorough seismological and geological investigation (see Ref. 1 for more detail).

Computational Steps

The computation of the earthquake response of the dam (spillway monolith plus pier) is organized in three parts:

Part I: The earthquake forces and stresses due to the fundamental vibration mode can be determined approximately for purposes of preliminary design by the following computational steps:

1. Compute T_1, the fundamental vibration period of the dam, in seconds, on rigid foundation rock with an empty reservoir from Eq. 6 in which H_s = total (monolith plus pier) height of the dam in feet, and E_s = design value for Young's modulus of elasticity of concrete, in pounds per square inch.
2. Compute $\tilde{T}_r$, the fundamental vibration period of the dam, in seconds, including the influence of impounded water from Eq. 4a in which T_1 was computed in Step 1; R_r = period ratio determined from Figs. 4 and 5 or Tables 2 and 3 for the design values of E_s, the wave reflection coefficient α, and the depth ratio H/H_s, where H is the depth of the impounded water, in feet. If $H/H_s < 0.5$, computation of R_r may be avoided by using $R_r \approx 1$. Values for R_r are presented for higher dams ($H_s \geq 300$ ft)) in Fig. 4 and Table 2, and for lower dams ($H_s < 300$ ft) in Fig. 5 and Table 3.
3. Compute the period ratio R_w from Eq. 7 in which $\tilde{T}r$ was computed in Step 2 and $T_1^r = 4H/C$, where $C = 4720$ feet per second.

4. Compute $\tilde{T}_1$, the fundamental vibration period the dam in seconds, including the influence of foundation flexibility and of impounded water, from Eq. 4c in which R_r was determined from Step 2; R_f = period ratio determined for the design value of E_f/E_s from Fig. 8 or Table 4 for dams with $H_s \geq 300$ ft or from Fig. 10 or Table 5 if $H_s < 300$ ft. If $E_f/E_s > 4$, use $R_f \approx 1$.

5. Compute the damping ratio $\tilde{\xi}_1$ of the dam from Eq. 5 using the period ratios R_r and R_f determined in Steps 2 and 4, respectively; ξ_1 = viscous damping ratio for the dam on rigid foundation rock with empty reservoir; ξ_r = added damping ratio due to dam-water interaction and reservoir bottom absorption, obtained from Fig. 6 or Table 2 for dams with $H_s \geq 300$ ft, or from Fig. 7 or Table 3 if $H_s < 300$ ft, for the selected values of E_s, α, and H/H_s; and ξ_f = added damping ratio due to dam-foundation rock interaction, obtained from Fig. 9 or Table 4 for dams with $H_s \geq 300$ ft, or Fig. 11 or Table 5 if $H_s < 300$ ft, for the design values of E_f/E_s and η_f. If $H/H_s < 0.5$, use $\xi_r = 0$; if $E_f/E_s > 4$, use $\xi_f = 0$; and if the computed value of $\tilde{\xi}_1 < \xi_1$, use $\tilde{\xi}_1 = \xi_1$.

6. Determine $gp(y,\tilde{T}_r)$ from Fig. 12 or Table 6 corresponding to the value of R_w computed in Step 3--rounded to one of the two nearest available values, the one giving the larger $p(y)$ --the design value of α, and for $H/H_s = 1$; the result is multiplied by $(H/H_s)^2$. If $H/H_s < 0.5$, computation of $p(y, \tilde{T}_r)$ may be avoided using $p(y, \tilde{T}_r) \approx 0$..

7. Compute the generalized mass $\tilde{M}_1$ from Eq. 8a, in which R_r was computed in Step 2, and M_1 is computed from Eq. 2b, in which $w_s(y)$ = the weight per unit height of the equivalent two-dimensional system of unit thickness representing the dam (see page 2); the fundamen-

tal vibration mode shape $\phi(y)$ is given in Fig. 3 or Table 1; and g = 32.2 feet per squared second. Evaluation of Eq. 2b may be avoided by obtaining an approximate value from $M_1 = 0.060\, W_s/g$, where W_s is the total weight of the equivalent two-dimensional system.

8. Compute the effective earthquake force coefficient $\tilde{L}_1$ from Eq. 8b in which L_1 is computed from Eq. 3b; $F_{st} = wH^2/2$; and A_p is given in Table 7 for the values of R_w and α used in Step 6. If $H/H_s < 0.5$, computation of $\tilde{L}_1$ may be avoided by using $\tilde{L}_1 \approx L_1$. Evaluation of Eq. 3b may be avoided by obtaining an approximate value from $L_1 = 0.17\, W_s/g$.

   Note: Computation of Steps 7 and 8 may be avoided by using conservative values: $\tilde{L}_1/\tilde{M}_1 = 4$ for dams with impounded water, and $L_1/M_1 = 3$ for dams with empty reservoirs.

9. Compute $f_1(y)$, the equivalent lateral earthquake forces associated with the fundamental vibration mode from Eq. 1 in which $S_a(\tilde{T}_1, \tilde{\xi}_1)$ = the pseudo-accleration ordinate of the earthquake design spectrum in feet per square second at period $\tilde{T}_1$ determined in Step 4 and damping ratio $\tilde{\xi}_1$ determined in Step 5; $w_s(y)$ was defined in Step 7; $\phi(y)$ = fundamental vibration mode shape of the dam from Fig. 3 or Table 1; $\tilde{M}_1$ and $\tilde{L}_1$ = generalized mass and earthquake coefficient determined in Steps 7 and 8, respectively; and the hydrodynamic pressure term $gp(y, \tilde{T}_r)$ was determined in Step 6; and g = 32.2 feet per squared second.

10. Determine by static analysis of the dam subject to equivalent lateral forces $f_1(y)$, from Step 9, applied to the upstream face of the dam, all the response quantities of interest, in particular the stresses throughout the dam. Traditional procedures for design calculations may be used wherein the direct and bending stresses across a horizontal section are computed by elementary formulas for stresses in beams.*

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*However, the beam theory overestimates the stresses near the sloped downstream face by a factor that depends on this slope and the heightwise distribution of equivalent lateral forces. A correction factor is recommended in the next section for the sloping part of the downstream face.

Part II: The earthquake forces and stresses due to the higher vibration modes can be detetermined approximately for purposes of preliminary design by the following computational steps:

11. Compute $f_{sc}(y)$, the lateral forces associated with the higher vibration modes from Eq. 9, in which M_1 and L_1 were determined in Steps 7 and 8, respectively; $g\,p_0(y)$ is determined from Fig. 13 or Table 8; B_1 is computed from Eq. 10; and a_g is the maximum ground acceleration, in feet per squared second, of the design earthquake. If $H/H_s < 0.5$, computation of $p_0(y)$ may be avoided by using $p_0(y) \approx 0$ and hence $B_1 \approx 0$.

12. Determine by static analysis of the dam subjected to the equivalent lateral forces $f_{sc}(y)$, from Step 11, applied to the upstream face of the dam dam, all the response quantities of interest, in particular the stresses throughout the dam. The stress analysis may be carried out by the procedures mentioned in Step 10.

Part III: The total earthquake forces and stresses in the dam are determined by the following computational step:

13. Compute the total value of any response quantity by Eq. 12, in which r_1 and r_{sc} are values of the response quantity determined in Steps 10 and 11 associated with the fundamental and higher vibration modes, respectively, and r_{st} is its initial value prior to the earthquake due to various loads, including the self-weight of the dam, hydrostatic pressure, and thermal effects.

Use of Metric Units

Because the standard values for most quantities required in the simplified analysis procedure are presented in non-dimensional form, implementation of the procedure in metric units is straightforward. The few expressions and data requiring conversion to metric units are noted next:

1. The fundamental vibration T_1 of the dam on rigid foundation rock with empty reservoir (Step 1), in seconds, is given by:

$$T_1 = 0.33\frac{H_s}{\sqrt{E_s}} \qquad \text{if} \quad H_s \geq 300\text{ ft} \tag{14a}$$

$$T_1 = 0.34\frac{H_s}{\sqrt{E_s}} \qquad \text{if} \quad H_s < 300\text{ ft} \tag{14b}$$

where H_s is the total (monolith plus pier) height of the dam in meters; and E_s is the Young's modulus of elasticity of the dam concrete in mega-Pascals.

2. The period ratio R_r and added damping ratio ξ_r due to dam-water interaction presented in Figs. 4 to 7 and Tables 2 and 3 is for specified values of E_s in psi which should be converted to mega-Pascals as follows: 1 million psi = 7 thousand mega-Pascals.

3. Where required in the calculations, the unit weight of water $w = 9.81$ kilo-Newtons per cubic meter; the acceleration due to gravity $g = 9.81$ meters per squared second; and the velocity of pressure waves in water $C = 1440$ meters per second.

EXAMPLE ANALYSES

System and Ground Motion

The tallest gated spillway monolith of Pine Flat Dam is shown in Fig. 14. In accordance with earlier conclusions, the effects of the gate, bridge, and foot bucket are neglected in this simplified analysis. The total (monolith plus pier) height of the dam $H_s = 400$ ft; monolith thickness = 50 ft, pier thickness = 8 ft, and the ratio of the two is 0.16; modulus of elasticity of concrete, $E_s = 3.25 \times 10^6$ psi; unit weight of concrete = 155 pcf; damping ratio, $\xi_1 = 5\%$; modulus of elasticity of foundation rock, $E_f = 3.25 \times 10^6$ psi; constant hysteretic damping coefficient of foundation rock, $\eta_f = 0.10$; depth of water, $H = 381$ ft; and, at the reservoir bottom, the wave reflection coefficient, $\alpha = 0.5$.

The dam is analyzed for the earthquake ground motion characterized by the smooth design spectrum of Fig. 15, scaled by a factor of 0.25. The spectrum of Fig. 15 is developed by well established procedures [14] for excitations with maximum ground acceleration a_g, velocity v_g, and displacement u_g of $1g$, 48 in./sec, and 36 in., respectively. Amplification factors for the acceleration-controlled, velocity-controlled, and displacement-controlled regions of the spectrum were taken from Ref. 14 for 84.1 percentile response.

Computation of Earthquake Forces

The dam is analyzed by the simplified analysis procedure for the four cases listed in Table 9. Implementation of the step-by-step analysis procedure in the preceding section is summarized next with additional details available in Appendix B; all computations are performed for the equivalent two-dimensional system of unit thickness (see page 2) representing the monolith and pier. Because the height H_s of Pine Flat Dam exceeds 300 ft, all the parameters in the subsequent computations are obtained from tables and figures presented for "higher" dams:

1. For $E_s = 3.25 \times 10^6$ psi and $H_s = 400$ ft, from Eq. 6a, $T_1 = (1.2)(400)/\sqrt{3.25 \times 10^6} = 0.266$ sec.

2. For $E_s = 3.0 \times 10^6$ psi (rounded down from 3.25×10^6 psi), $\alpha = 0.50$ and $H/H_s = 381/400 = 0.95$, Fig. 4(e) or Table 2(e) gives $R_r = 1.319$, so $\tilde{T}_r = (1.319)(0.266) = 0.351$ sec.

3. From Eq. 7, $T_1^r = (4)(381)/4720 = 0.323$ sec and $R_w = 0.323/0.351 = 0.92$.

4. For $E_f/E_s = 1$, Fig. 8 or Table 4 gives $R_f = 1.224$, so $\tilde{T}_1 = (1.224)(0.266) = 0.326$ sec for Case 3, and $\tilde{T}_1 = (1.224)(0.351) = 0.429$ sec for Case 4.

5. For Cases 2 and 4, $\xi_r = 0.046$ from Fig. 6(e) or Table 2(b) for $E_s = 3.0 \times 10^6$ psi (rounded down from 3.25×10^6 psi), $\alpha = 0.50$, and $H/H_s = 0.95$. For Cases 3 and 4, $\xi_f = 0.091$ from

Fig. 9 or Table 4 for $E_f/E_s = 1$ and $\eta_f = 0.10$. With $\xi_1 = 0.05$, Eq. 5 gives: $\tilde{\xi}_1 = (0.05)/(1.319) + 0.046 = 0.084$ for Case 2; $\tilde{\xi}_1 = (0.05)/(1.224)^3 + 0.091 = 0.118$ for Case 3; and $\tilde{\xi}_1 = (0.05)/[(1.319)(1.224)^3] + 0.046 + 0.091 = 0.158$ for Case 4.

6. The values of $gp(y)$ are obtained at fifteen levels (Fig. 16) from Fig. 12(d) or Table 6(d) for $R_w = 0.90$ (by rounding $R_w = 0.92$ from Step 3 and $\alpha = 0.50$, and multiplied by $(0.0624)(381)(0.95)^2 = 21.5$ kip/ft.

7. Evaluating Eq. 2b in discrete form gives $M_1 = (1/g)(559\text{ kip})$. From Eq. 8a, $\tilde{M}_1 = (1.319)^2(1/g)(559) = (973\text{ kip})/g$.

8. Equation 3b in discrete form gives $L_1 = (1623\text{ kip})/g$. From Table 7(b), $A_p = 0.351$ for $R_w = 0.90$ and $\alpha = 0.50$. Equation 8b then gives $\tilde{L}_1 = 1623/g + [(0.0624)(381)^2/2g](0.95)^2(0.351) = (3059\text{ kip})/g$. Consequently, for Cases 1 and 3, $L_1/M_1 = 1623/559 = 2.90$, and for Cases 2 and 4, $\tilde{L}_1/\tilde{M}_1 = 3059/973 = 3.14$.

9. For each of the four cases, Eq. 1 was evaluated to obtain the equivalent lateral forces $f_1(y)$ at fifteen locations along the height of the dam (Fig. 16), including the top and bottom, by substituting values for the quantities computed in the preceding steps; computing the weight $w_s(y)$ per unit height of the monolith from the monolith dimensions (Fig. 14) and the unit weight of concrete; by computing the weight $w_s(y)$ per unit height of the pier from the pier dimensions and the unit weight of concrete, multiplied by 0.16; and by substituting $\phi(y)$ from Fig. 3 or Table 1 and the $S_a(\tilde{T}_1, \tilde{\xi}_1)$ from Fig. 15 corresponding to $\tilde{T}_1$ and $\tilde{\xi}_1$ obtained in Steps 4 and 5 (Table 9). The resulting equivalent lateral forces $f_1(y)$ are presented in Table 10 for each case.

10. The static stress analysis of the dam subjected to the equivalent lateral forces $f_1(y)$, from Step 9, applied to the upstream face of the dam, is described in the next subsection, leading to response value r_1 at a particular location in the dam.
11. For each of the four cases, Eq. 9 was evaluated to obtain the equivalent lateral forces $f_{sc}(y)$ at fifteen locations along the height of the dam (Fig. 16), including the top and bottom, by substituting numerical values for the quantities computed in the preceding steps; obtaining $g\,p_0(y)$ from Fig. 13 or Table 8; using Eq. 10 to compute $B_1 = 0.25[(0.0624)(381)^2/2g](0.95)^2 =$ $(1027\text{ kip})/g$, leading to $B_1/M_1 = 1027/559 = 1.837$; and substituting $a_g = 0.25\,g$. The resulting equivalent lateral forces $f_{sc}(y)$ are presented in Table 10 for each case.
12. The static stress analysis of the dam subjected to the equivalent lateral forces $f_{sc}(y)$, from Step 11, applied to the upstream face of the dam, is described in the next subsection, leading to response value r_{sc} at a particular location in the dam.
13. Compute the maximum total value of any response quantity by combining r_1 from Step 10, r_{sc} from Step 12, and r_{st}, the initial value prior to the earthquake, according to Eq. 12; this is described further in the next subsection.

Computation of Stresses

The equivalent lateral earthquake forces $f_1(y)$ and $f_{sc}(y)$ representing the maximum effects of the fundamental and higher vibration modes, respectively, were computed in Steps 9 and 11. Dividing the dam into fourteen blocks shown in Fig. 16, each of these sets of distributed forces is replaced by statically equivalent concentrated forces at the centroids of the blocks. Considering the dam monolith to be a cantilever beam, the vertical bending stresses are computed at the bottom of the blocks of the monolith only (not the pier) using elementary formulas for stresses in beams. The resulting vertical bending stresses due to the fundamental vibration mode and the higher vibration modes are presented in Table 11 for the four analysis cases. Also included are the combined values obtained by the SRSS combination rule. These stresses occur at the upstream face when the earthquake forces act in the downstream direction, and at the downstream face when the earthquake forces act in the upstream direction. In this simple stress analysis the foundation rock is implicitly assumed to be rigid. These computations are implemented by a modified version of the computer program presented in Ref. 2. A description and listing of the computer program is included in this report (Appendix C).

This procedure for computing the stresses is not implemented for the pier. Instead, it should be analyzed as a reinforced concrete structure for the lateral forces computed above. Furthermore, it may be necessary to include the effects of ground motion along the dam axis in the pier analysis.

The vertical bending stresses can be transformed to principal stresses, as described in Appendix C of Ref. 2. Because the upstream face of Pine Flat Dam is nearly vertical and the effects of the tail water at the downstream face are negligible, as shown in Appendix C of Ref. 2, the principal stresses σ_1 and σ_{sc} at any location in the dam due to the forces $f_1(y)$ and $f_{sc}(y)$, respectively, may be combined using the SRSS combination rule, Eq. 11 [2].

As shown in Refs. 2 and 3 for nonoverflow monoliths, while the simplified procedure provides excellent estimates of the maximum stress on the upstream face, at the same time it significantly overestimates the maximum stress on the downstream face. This discrepancy is due primarily to the limitations of elementary beam theory in predicting stresses near sloped faces. The beam theory overestimates the stresses near the sloped downstream face by a factor that depends on this slope and the heightwise distribution of equivalent lateral forces. Based on a comparison of results from beam theory and finite element analysis, it is recommended that σ_{y1} and $\sigma_{y,sc}$ computed at the sloping part of the downstream face by beam theory (Steps 10 and 12) should be multiplied by a factor of 0.8. Similarly, the beam theory is incapable of reproducing the stress concentration in the heel area of dams predicted by the refined analysis [2,3]; so the stresses in that area are underestimated. These limitations can be overcome by using the finite element method for static analysis of the dam in Steps 10 and 12 of "Computational Steps."

CONCLUSION

A simplified procedure was presented in 1986 [2] for earthquake analysis of concrete gravity dams. Developed in a form appropriate for preliminary design and safety evaluation of dams, this procedure was presented specifically for nonoverflow monoliths. In this report, the analysis procedure has been extended to gated spillway monoliths, the standard data required for the analysis of such structures has been presented in the form of design charts and tables, and a computer program has been made available to facilitate implementation of the procedure.

This procedure is suitable for stress analysis of the spillway monolith but not for the pier. The latter should be analyzed as a reinforced concrete structure for the lateral forces associated with upstream-downstream ground motion, computed by the procedure presented in this report, and the forces associated with earthquake motion along the dam axis. Evaluation of the latter set of forces is beyond the scope of this report.

Similarly, refined response history analysis procedures [7] are not presently available for analysis of piers. Such procedures could be useful in the seismic safety evaluation of existing dams if the simplified analysis indicates that the piers are likely to be damaged.

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NOTATION

The following symbols are used in this report:

A_p = integral of $2gp(\hat{y})/wH$ over depth of the impounded water for $H/H_s = 1$ as listed in Table 7

a_g = maximum ground acceleration

B_1 = defined in Eq. 10

C = velocity of pressure waves in water

E_f = Young's modulus of elasticity of foundation rock

E_s = Young's modulus of elasticity of dam concrete

F_{st} = $wH^2/2$

$f_1(y)$ = equivalent lateral forces on the upstream face of the dam due to the fundamental vibration mode, as defined in Eq. 1

$f_{sc}(y)$ = equivalent lateral forces on the upstream face of the dam due to higher vibration modes, as defined in Eq. 9

g = acceleration due to gravity

H = depth of impounded water

H_s = height of upstream face of dam (monolith plus pier)

L_1 = integral defined in Eq. 3b

$\tilde{L}_1$ = defined in Eq. 3a

M_1 = integral defined in Eq. 2b

$\tilde{M}_1$ = defined in Eq. 2a

$p_0(y)$ = hydrodynamic pressure on a rigid dam with water compressibility neglected

$p(y, \tilde{T}_r)$	=	$Re[\bar{p}_1(y, \tilde{T}_r)]$
$\bar{p}_1(y, \tilde{T}_r)$	=	complex-valued hydrodynamic pressure on the upstream face due to harmonic acceleration of dam, at period $\tilde{T}_r$, in the fundamental vibration mode
R_f	=	period lengthening ratio due to foundation-rock flexibility effects
R_r	=	period lengthening ratio due to hydrodynamic effects
R_w	=	$T_1^r/\tilde{T}_r$
r_1	=	maximum response due to the fundamental vibration mode
r_d	=	maximum dynamic response
r_{max}	=	maximum total response of dam
r_{sc}	=	maximum response due to the higher vibration modes
r_{st}	=	response due to initial static effects
$S_a(\tilde{T}_1, \xi_1)$	=	ordinate of pseudo-acceleration response spectrum for the ground motion evaluated at period $\tilde{T}_1$ and damping ratio ξ_1
T_1	=	fundamental vibration period of dam on rigid foundation rock with empty reservoir given by Eq. 6
$\tilde{T}_1$	=	fundamental resonant period of dam on flexible foundation rock with impounded water given by Eq. 4c
T_1^r	=	$4H/C$, fundamental vibration period of impounded water
$\tilde{T}_f$	=	fundamental resonant period of dam on flexible foundation rock with empty reservoir given by Eq. 4b
$\tilde{T}_r$	=	fundamental resonant period of dam on rigid foundation rock with impounded water given by Eq. 4a
W_s	=	total weight of dam
w	=	unit weight of water

$w_s(y)$	=	weight of dam per unit height; the actual weight of the monolith should be used, but for the pier it should be divided by the ratio of monolith thickness to pier thickness
y	=	coordinate along the height of the dam
$\hat{y}$	=	y/H
α	=	wave reflection coefficient for reservoir bottom materials or foundation rock
η_f	=	constant hysteretic damping factor for foundation rock
ξ_1	=	damping ratio of dam on rigid foundation rock with empty reservoir
$\tilde{\xi}_1$	=	damping ratio for dam on flexible foundation rock with impounded water
ξ_f	=	added damping ratio due to foundation-rock flexibility effects
ξ_r	=	added damping ratio due to hydrodynamic effects
$\phi(y)$	=	fundamental vibration mode shape of dam at upstream face

TABLES

Table 1(a) -- Standard Fundamental Mode Shape of Vibration for Spillways of Concrete Gravity Dams -- *Higher Dams*

y/H_s	$\phi(y)$
1.0	1.000
.95	.909
.90	.816
.85	.725
.80	.646
.75	.572
.70	.504
.65	.440
.60	.381
.55	.327
.50	.277
.45	.232
.40	.192
.35	.155
.30	.123
.25	.094
.20	.070
.15	.048
.10	.030
.05	.016
0.	0.

Table 1(b) -- Standard Fundamental Mode Shape of Vibration for Spillways of Concrete Gravity Dams -- *Lower Dams*

y/H_s	$\phi(y)$
1.0	1.000
.95	.914
.90	.825
.85	.738
.80	.654
.75	.574
.70	.499
.65	.440
.60	.386
.55	.334
.50	.285
.45	.239
.40	.197
.35	.160
.30	.126
.25	.096
.20	.070
.15	.048
.10	.030
.05	.015
0.	0.

Table 2(a) -- Standard Values for R_r and ξ_r, the Period Lengthening Ratio and Added Damping Ratio due to Hydrodynamic Effects, for Modulus of Elasticity of Concrete, E_s = 5, 4.5 and 4 million psi. -- *Higher Dams*

H/H_s	α	$E_s = 5 \times 10^6$ psi		$E_s = 4.5 \times 10^6$ psi		$E_s = 4 \times 10^6$ psi	
		R_r	ξ_r	R_r	ξ_r	R_r	ξ_r
1.0	1.0	1.642	0.	1.582	0.	1.524	0.
	.9	1.653	0.062	1.590	0.049	1.529	0.036
	.75	1.650	0.073	1.585	0.070	1.527	0.061
	.5	1.502	0.084	1.484	0.079	1.458	0.072
	.25	1.364	0.075	1.366	0.071	1.368	0.065
	.0	1.333	0.055	1.332	0.052	1.330	0.048
.95	1.0	1.548	0.	1.488	0.	1.433	0.
	.9	1.560	0.062	1.497	0.051	1.439	0.037
	.75	1.553	0.068	1.493	0.065	1.435	0.058
	.5	1.350	0.083	1.368	0.075	1.359	0.067
	.25	1.277	0.066	1.280	0.062	1.284	0.057
	.0	1.258	0.047	1.256	0.044	1.255	0.041
.90	1.0	1.460	0.	1.401	0.	1.348	0.
	.9	1.471	0.059	1.410	0.049	1.353	0.036
	.75	1.460	0.061	1.401	0.060	1.348	0.054
	.5	1.235	0.075	1.267	0.068	1.269	0.060
	.25	1.208	0.055	1.212	0.052	1.214	0.048
	.0	1.195	0.039	1.195	0.036	1.193	0.034
.85	1.0	1.374	0.	1.318	0.	1.267	0.
	.9	1.385	0.053	1.326	0.045	1.276	0.031
	.75	1.368	0.055	1.314	0.054	1.267	0.047
	.5	1.170	0.062	1.192	0.057	1.198	0.050
	.25	1.155	0.044	1.157	0.041	1.159	0.038
	.0	1.146	0.031	1.144	0.029	1.144	0.027
.80	1.0	1.290	0.	1.239	0.	1.196	0.
	.9	1.300	0.046	1.248	0.037	1.203	0.024
	.75	1.277	0.049	1.232	0.046	1.195	0.038
	.5	1.126	0.048	1.139	0.044	1.143	0.038
	.25	1.114	0.034	1.116	0.031	1.117	0.029
	.0	1.106	0.023	1.106	0.022	1.105	0.020
.75	1.0	1.209	0.	1.166	0.	1.134	0.
	.9	1.220	0.037	1.175	0.027	1.139	0.016
	.75	1.188	0.042	1.159	0.036	1.134	0.027
	.5	1.094	0.036	1.100	0.032	1.101	0.027
	.25	1.082	0.024	1.083	0.023	1.085	0.020
	.0	1.076	0.017	1.076	0.016	1.075	0.015

Table 2(a) -- Continued

H/H_s	α	$E_s = 5 \times 10^6$ psi		$E_s = 4.5 \times 10^6$ psi		$E_s = 4 \times 10^6$ psi	
		R_r	ξ_r	R_r	ξ_r	R_r	ξ_r
.70	1.0	1.135	0.	1.104	0.	1.086	0.
	.9	1.144	0.026	1.110	0.016	1.089	0.008
	.75	1.117	0.032	1.101	0.024	1.087	0.016
	.5	1.070	0.024	1.072	0.021	1.071	0.017
	.25	1.058	0.017	1.059	0.015	1.059	0.014
	.0	1.054	0.012	1.054	0.011	1.054	0.010
.65	1.0	1.074	0.	1.060	0.	1.054	0.
	.9	1.081	0.013	1.064	0.007	1.055	0.004
	.75	1.071	0.018	1.062	0.012	1.054	0.008
	.5	1.049	0.015	1.049	0.012	1.047	0.010
	.25	1.041	0.011	1.042	0.010	1.042	0.009
	.0	1.037	0.008	1.037	0.007	1.037	0.007
.60	1.0	1.040	0.	1.036	0.	1.033	0.
	.9	1.042	0.004	1.037	0.002	1.034	0.001
	.75	1.041	0.008	1.036	0.005	1.034	0.003
	.5	1.033	0.008	1.032	0.007	1.031	0.005
	.25	1.028	0.007	1.028	0.006	1.028	0.005
	.0	1.026	0.005	1.026	0.005	1.026	0.004
.55	1.0	1.024	0.	1.022	0.	1.020	0.
	.9	1.024	0.001	1.023	0.001	1.022	0.001
	.75	1.024	0.003	1.023	0.002	1.022	0.002
	.5	1.022	0.004	1.020	0.003	1.020	0.003
	.25	1.019	0.004	1.018	0.003	1.018	0.003
	.0	1.017	0.003	1.017	0.003	1.017	0.003
.50	1.0	1.014	0.	1.013	0.	1.013	0.
	.9	1.014	0.000	1.013	0.000	1.013	0.000
	.75	1.014	0.001	1.014	0.001	1.013	0.001
	.5	1.013	0.002	1.013	0.002	1.013	0.001
	.25	1.012	0.002	1.012	0.002	1.012	0.002
	.0	1.011	0.002	1.011	0.002	1.011	0.002

Table 2(b) -- Standard Values for R_r and ξ_r, the Period Lengthening Ratio and Added Damping Ratio due to Hydrodynamic Effects, for Modulus of Elasticity of Concrete, E_s = 3.5 and 3 million psi. -- *Higher Dams*

H/H_s	α	$E_s = 3.5 \times 10^6$ psi		$E_s = 3 \times 10^6$ psi	
		R_r	ξ_r	R_r	ξ_r
1.0	1.0	1.475	0.	1.433	0.
	.9	1.477	0.024	1.435	0.015
	.75	1.475	0.048	1.433	0.034
	.5	1.433	0.062	1.407	0.051
	.25	1.364	0.058	1.361	0.051
	.0	1.326	0.045	1.325	0.042
.95	1.0	1.383	0.	1.344	0.
	.9	1.389	0.024	1.348	0.014
	.75	1.385	0.046	1.346	0.032
	.5	1.339	0.058	1.319	0.046
	.25	1.282	0.051	1.279	0.045
	.0	1.253	0.039	1.252	0.035
.90	1.0	1.302	0.	1.267	0.
	.9	1.307	0.022	1.271	0.013
	.75	1.304	0.041	1.269	0.028
	.5	1.259	0.050	1.245	0.039
	.25	1.214	0.043	1.212	0.037
	.0	1.192	0.032	1.191	0.029
.85	1.0	1.229	0.	1.201	0.
	.9	1.233	0.019	1.203	0.010
	.75	1.230	0.035	1.202	0.023
	.5	1.192	0.041	1.183	0.031
	.25	1.160	0.034	1.159	0.029
	.0	1.143	0.025	1.143	0.023
.80	1.0	1.166	0.	1.144	0.
	.9	1.170	0.013	1.147	0.007
	.75	1.167	0.026	1.146	0.016
	.5	1.139	0.031	1.133	0.023
	.25	1.117	0.025	1.116	0.022
	.0	1.105	0.019	1.105	0.017
.75	1.0	1.114	0.	1.101	0.
	.9	1.116	0.008	1.103	0.004
	.75	1.115	0.017	1.101	0.011
	.5	1.099	0.021	1.094	0.016
	.25	1.083	0.018	1.083	0.015
	.0	1.075	0.014	1.075	0.013

Table 2(b) -- Continued

H/H_s	α	$E_s = 3.5 \times 10^6$ psi		$E_s = 3 \times 10^6$ psi	
		R_r	ξ_r	R_r	ξ_r
.70	1.0	1.075	0.	1.068	0.
	.9	1.076	0.004	1.070	0.003
	.75	1.076	0.010	1.070	0.006
	.5	1.068	0.013	1.065	0.010
	.25	1.059	0.012	1.058	0.010
	.0	1.054	0.009	1.054	0.009
.65	1.0	1.048	0.	1.046	0.
	.9	1.049	0.002	1.045	0.001
	.75	1.049	0.005	1.044	0.004
	.5	1.046	0.008	1.041	0.006
	.25	1.041	0.008	1.038	0.007
	.0	1.037	0.006	1.035	0.006
.60	1.0	1.031	0.	1.029	0.
	.9	1.031	0.001	1.029	0.001
	.75	1.032	0.002	1.028	0.002
	.5	1.030	0.004	1.027	0.003
	.25	1.028	0.005	1.025	0.004
	.0	1.026	0.004	1.023	0.004
.55	1.0	1.020	0.	1.018	0.
	.9	1.020	0.000	1.018	0.000
	.75	1.020	0.001	1.018	0.001
	.5	1.019	0.002	1.017	0.002
	.25	1.018	0.003	1.016	0.002
	.0	1.017	0.002	1.015	0.002
.50	1.0	1.013	0.	1.011	0.
	.9	1.013	0.000	1.011	0.000
	.75	1.013	0.001	1.011	0.000
	.5	1.013	0.001	1.010	0.001
	.25	1.012	0.001	1.010	0.001
	.0	1.011	0.001	1.009	0.001

Table 2(c) -- Standard Values for R_r and ξ_r, the Period Lengthening Ratio and Added Damping Ratio due to Hydrodynamic Effects, for Modulus of Elasticity of Concrete, E_s = 2.5 and 2 million psi.
Higher Dams

H/H_s	α	$E_s = 2.5 \times 10^6$ psi		$E_s = 2 \times 10^6$ psi	
		R_r	ξ_r	R_r	ξ_r
1.0	1.0	1.399	0.	1.384	0.
	.9	1.401	0.009	1.384	0.006
	.75	1.401	0.023	1.380	0.015
	.5	1.385	0.039	1.365	0.029
	.25	1.353	0.043	1.341	0.037
	.0	1.323	0.038	1.319	0.035
.95	1.0	1.314	0.	1.301	0.
	.9	1.316	0.009	1.301	0.005
	.75	1.316	0.021	1.297	0.013
	.5	1.302	0.035	1.284	0.026
	.25	1.274	0.037	1.264	0.032
	.0	1.250	0.032	1.246	0.030
.90	1.0	1.242	0.	1.230	0.
	.9	1.244	0.007	1.230	0.004
	.75	1.242	0.018	1.227	0.011
	.5	1.232	0.029	1.217	0.022
	.25	1.209	0.031	1.201	0.026
	.0	1.191	0.026	1.187	0.025
.85	1.0	1.181	0.	1.172	0.
	.9	1.182	0.006	1.172	0.003
	.75	1.182	0.014	1.170	0.009
	.5	1.172	0.023	1.162	0.017
	.25	1.156	0.024	1.150	0.021
	.0	1.142	0.021	1.140	0.019
.80	1.0	1.131	0.	1.125	0.
	.9	1.133	0.004	1.125	0.002
	.75	1.133	0.010	1.123	0.006
	.5	1.126	0.017	1.118	0.012
	.25	1.115	0.018	1.110	0.015
	.0	1.104	0.016	1.102	0.015
.75	1.0	1.095	0.	1.088	0.
	.9	1.095	0.003	1.088	0.002
	.75	1.093	0.007	1.087	0.004
	.5	1.087	0.012	1.084	0.008
	.25	1.079	0.014	1.079	0.011
	.0	1.073	0.012	1.073	0.011

Table 2(c) -- Continued

H/H_s	α	$E_s = 2.5 \times 10^6$ psi		$E_s = 2 \times 10^6$ psi	
		R_r	ξ_r	R_r	ξ_r
.70	1.0	1.065	0.	1.061	0.
	.9	1.064	0.002	1.061	0.001
	.75	1.063	0.004	1.060	0.003
	.5	1.060	0.008	1.058	0.005
	.25	1.055	0.009	1.055	0.007
	.0	1.051	0.008	1.051	0.007
.65	1.0	1.043	0.	1.040	0.
	.9	1.043	0.001	1.040	0.001
	.75	1.042	0.002	1.040	0.002
	.5	1.040	0.005	1.039	0.003
	.25	1.038	0.006	1.037	0.005
	.0	1.035	0.006	1.035	0.005
.60	1.0	1.027	0.	1.026	0.
	.9	1.027	0.001	1.026	0.000
	.75	1.027	0.001	1.026	0.001
	.5	1.026	0.003	1.026	0.002
	.25	1.025	0.003	1.025	0.003
	.0	1.023	0.004	1.023	0.003
.55	1.0	1.017	0.	1.016	0.
	.9	1.017	0.000	1.016	0.000
	.75	1.017	0.001	1.016	0.001
	.5	1.017	0.001	1.016	0.001
	.25	1.016	0.002	1.016	0.002
	.0	1.015	0.002	1.015	0.002
.50	1.0	1.010	0.	1.010	0.
	.9	1.010	0.000	1.010	0.000
	.75	1.010	0.000	1.010	0.000
	.5	1.010	0.001	1.010	0.001
	.25	1.010	0.001	1.010	0.001
	.0	1.009	0.001	1.009	0.001

Table 3(a) -- Standard Values for R_r and ξ_r, the Period Lengthening Ratio and Added Damping Ratio due to Hydrodynamic Effects, for Modulus of Elasticity of Concrete, E_s = 5, 4.5 and 4 million psi. -- *Lower Dams*

H/H_s	α	$E_s = 5 \times 10^6$ psi		$E_s = 4.5 \times 10^6$ psi		$E_s = 4 \times 10^6$ psi	
		R_r	ξ_r	R_r	ξ_r	R_r	ξ_r
1.0	1.0	1.647	0.	1.600	0.	1.555	0.
	.9	1.653	0.043	1.605	0.031	1.560	0.022
	.75	1.653	0.069	1.603	0.059	1.558	0.048
	.5	1.572	0.081	1.543	0.074	1.518	0.065
	.25	1.454	0.075	1.451	0.070	1.447	0.063
	.0	1.405	0.057	1.403	0.054	1.401	0.050
.95	1.0	1.546	0.	1.497	0.	1.456	0.
	.9	1.553	0.046	1.504	0.034	1.458	0.024
	.75	1.550	0.067	1.499	0.059	1.456	0.048
	.5	1.456	0.077	1.435	0.070	1.412	0.062
	.25	1.350	0.068	1.350	0.063	1.348	0.057
	.0	1.316	0.050	1.314	0.047	1.312	0.044
.90	1.0	1.451	0.	1.405	0.	1.362	0.
	.9	1.458	0.047	1.410	0.034	1.368	0.023
	.75	1.454	0.063	1.407	0.056	1.364	0.045
	.5	1.346	0.072	1.335	0.065	1.321	0.056
	.25	1.266	0.059	1.266	0.055	1.266	0.049
	.0	1.241	0.042	1.239	0.040	1.238	0.037
.85	1.0	1.361	0.	1.318	0.	1.280	0.
	.9	1.370	0.044	1.325	0.032	1.285	0.021
	.75	1.362	0.057	1.318	0.051	1.282	0.039
	.5	1.250	0.063	1.250	0.056	1.241	0.048
	.25	1.198	0.048	1.199	0.045	1.199	0.041
	.0	1.179	0.034	1.179	0.032	1.178	0.030
.80	1.0	1.277	0.	1.238	0.	1.208	0.
	.9	1.287	0.038	1.245	0.027	1.212	0.016
	.75	1.276	0.050	1.238	0.043	1.209	0.032
	.5	1.179	0.052	1.181	0.045	1.175	0.038
	.25	1.144	0.038	1.146	0.035	1.146	0.031
	.0	1.133	0.026	1.131	0.025	1.131	0.023
.75	1.0	1.201	0.	1.168	0.	1.146	0.
	.9	1.208	0.030	1.174	0.019	1.149	0.011
	.75	1.195	0.041	1.168	0.032	1.147	0.023
	.5	1.127	0.039	1.129	0.033	1.125	0.027
	.25	1.105	0.028	1.105	0.026	1.105	0.023
	.0	1.095	0.019	1.095	0.018	1.094	0.017

Table 3(a) -- Continued

H/H_s	α	$E_s = 5 \times 10^6$ psi		$E_s = 4.5 \times 10^6$ psi		$E_s = 4 \times 10^6$ psi	
		R_r	ξ_r	R_r	ξ_r	R_r	ξ_r
.70	1.0	1.131	0.	1.111	0.	1.098	0.
	.9	1.138	0.020	1.115	0.011	1.100	0.006
	.75	1.129	0.029	1.111	0.021	1.099	0.014
	.5	1.091	0.026	1.089	0.022	1.087	0.018
	.25	1.075	0.019	1.075	0.018	1.075	0.016
	.0	1.067	0.014	1.067	0.013	1.067	0.012
.65	1.0	1.079	0.	1.070	0.	1.064	0.
	.9	1.083	0.009	1.072	0.005	1.065	0.003
	.75	1.080	0.016	1.071	0.011	1.064	0.007
	.5	1.063	0.016	1.060	0.013	1.058	0.011
	.25	1.053	0.013	1.052	0.011	1.052	0.010
	.0	1.047	0.009	1.047	0.009	1.047	0.008
.60	1.0	1.046	0.	1.043	0.	1.041	0.
	.9	1.048	0.003	1.044	0.002	1.041	0.001
	.75	1.047	0.007	1.044	0.005	1.041	0.004
	.5	1.041	0.009	1.040	0.007	1.038	0.006
	.25	1.035	0.008	1.035	0.007	1.035	0.006
	.0	1.032	0.006	1.032	0.006	1.032	0.005
.55	1.0	1.028	0.	1.027	0.	1.026	0.
	.9	1.029	0.001	1.027	0.001	1.026	0.001
	.75	1.029	0.003	1.027	0.002	1.026	0.002
	.5	1.027	0.004	1.026	0.004	1.025	0.003
	.25	1.024	0.004	1.024	0.004	1.024	0.003
	.0	1.022	0.004	1.022	0.003	1.022	0.003
.50	1.0	1.017	0.	1.016	0.	1.016	0.
	.9	1.017	0.001	1.016	0.000	1.016	0.000
	.75	1.017	0.001	1.016	0.001	1.016	0.001
	.5	1.016	0.002	1.016	0.002	1.016	0.001
	.25	1.015	0.002	1.015	0.002	1.015	0.002
	.0	1.014	0.002	1.014	0.002	1.014	0.002

Table 3(b) -- Standard Values for R_r and ξ_r, the Period Lengthening Ratio and Added Damping Ratio due to Hydrodynamic Effects, for Modulus of Elasticity of Concrete, E_s = 3.5 and 3 million psi. -- *Lower Dams*

H/H_s	α	$E_s = 3.5 \times 10^6$ psi		$E_s = 3 \times 10^6$ psi	
		R_r	ξ_r	R_r	ξ_r
1.0	1.0	1.520	0.	1.486	0.
	.9	1.522	0.015	1.488	0.011
	.75	1.520	0.036	1.488	0.026
	.5	1.493	0.055	1.471	0.044
	.25	1.441	0.056	1.433	0.049
	.0	1.399	0.047	1.395	0.043
.95	1.0	1.418	0.	1.389	0.
	.9	1.421	0.016	1.391	0.010
	.75	1.421	0.035	1.389	0.025
	.5	1.393	0.051	1.374	0.041
	.25	1.344	0.051	1.339	0.044
	.0	1.311	0.041	1.309	0.038
.90	1.0	1.330	0.	1.304	0.
	.9	1.332	0.015	1.306	0.009
	.75	1.332	0.033	1.304	0.023
	.5	1.304	0.046	1.289	0.036
	.25	1.264	0.044	1.259	0.038
	.0	1.236	0.035	1.236	0.032
.85	1.0	1.252	0.	1.230	0.
	.9	1.255	0.013	1.232	0.008
	.75	1.253	0.028	1.230	0.019
	.5	1.229	0.039	1.218	0.030
	.25	1.198	0.036	1.195	0.031
	.0	1.178	0.028	1.177	0.026
.80	1.0	1.185	0.	1.168	0.
	.9	1.188	0.010	1.170	0.006
	.75	1.186	0.022	1.170	0.014
	.5	1.168	0.030	1.160	0.023
	.25	1.146	0.028	1.143	0.024
	.0	1.130	0.022	1.130	0.020
.75	1.0	1.130	0.	1.120	0.
	.9	1.133	0.007	1.121	0.004
	.75	1.131	0.015	1.121	0.010
	.5	1.120	0.021	1.115	0.016
	.25	1.105	0.020	1.104	0.017
	.0	1.094	0.016	1.094	0.015

Table 3(b) -- Continued

H/H_s	α	$E_s = 3.5 \times 10^6$ psi		$E_s = 3 \times 10^6$ psi	
		R_r	ξ_r	R_r	ξ_r
.70	1.0	1.089	0.	1.082	0.
	.9	1.091	0.004	1.083	0.002
	.75	1.089	0.009	1.083	0.006
	.5	1.083	0.014	1.080	0.010
	.25	1.074	0.014	1.073	0.012
	.0	1.067	0.011	1.067	0.010
.65	1.0	1.059	0.	1.056	0.
	.9	1.059	0.002	1.056	0.001
	.75	1.059	0.005	1.055	0.004
	.5	1.056	0.008	1.052	0.007
	.25	1.052	0.009	1.048	0.008
	.0	1.047	0.008	1.044	0.007
.60	1.0	1.038	0.	1.036	0.
	.9	1.038	0.001	1.036	0.001
	.75	1.038	0.003	1.035	0.002
	.5	1.037	0.005	1.034	0.004
	.25	1.034	0.005	1.032	0.005
	.0	1.032	0.005	1.029	0.005
.55	1.0	1.025	0.	1.022	0.
	.9	1.025	0.001	1.022	0.000
	.75	1.025	0.001	1.022	0.001
	.5	1.025	0.002	1.022	0.002
	.25	1.023	0.003	1.020	0.003
	.0	1.022	0.003	1.019	0.003
.50	1.0	1.015	0.	1.013	0.
	.9	1.015	0.000	1.013	0.000
	.75	1.015	0.001	1.013	0.001
	.5	1.015	0.001	1.013	0.001
	.25	1.015	0.002	1.012	0.002
	.0	1.014	0.002	1.012	0.002

Table 3(c) -- Standard Values for R_r and ξ_r, the Period Lengthening Ratio and Added Damping Ratio due to Hydrodynamic Effects, for Modulus of Elasticity of Concrete, E_s = 2.5 and 2 million psi.
Lower Dams

H/H_s	α	$E_s = 2.5 \times 10^6$ psi		$E_s = 2 \times 10^6$ psi	
		R_r	ξ_r	R_r	ξ_r
1.0	1.0	1.460	0.	1.449	0.
	.9	1.462	0.007	1.448	0.005
	.75	1.462	0.018	1.446	0.013
	.5	1.451	0.034	1.433	0.026
	.25	1.425	0.041	1.412	0.035
	.0	1.393	0.039	1.387	0.037
.95	1.0	1.364	0.	1.354	0.
	.9	1.366	0.007	1.353	0.004
	.75	1.366	0.017	1.351	0.012
	.5	1.355	0.031	1.340	0.024
	.25	1.333	0.037	1.321	0.032
	.0	1.306	0.035	1.301	0.032
.90	1.0	1.282	0.	1.272	0.
	.9	1.284	0.006	1.272	0.004
	.75	1.284	0.015	1.270	0.010
	.5	1.274	0.027	1.261	0.020
	.25	1.255	0.032	1.246	0.027
	.0	1.235	0.029	1.230	0.027
.85	1.0	1.214	0.	1.205	0.
	.9	1.214	0.005	1.205	0.003
	.75	1.214	0.012	1.203	0.008
	.5	1.206	0.022	1.196	0.017
	.25	1.192	0.026	1.184	0.022
	.0	1.175	0.023	1.172	0.022
.80	1.0	1.156	0.	1.150	0.
	.9	1.157	0.004	1.150	0.002
	.75	1.157	0.009	1.149	0.006
	.5	1.152	0.017	1.144	0.013
	.25	1.142	0.020	1.135	0.017
	.0	1.130	0.018	1.126	0.017
.75	1.0	1.111	0.	1.107	0.
	.9	1.112	0.003	1.107	0.002
	.75	1.112	0.006	1.106	0.004
	.5	1.109	0.012	1.103	0.009
	.25	1.101	0.014	1.097	0.012
	.0	1.094	0.013	1.091	0.012

Table 3(c) -- Continued

H/H_s	α	$E_s = 2.5 \times 10^6$ psi		$E_s = 2 \times 10^6$ psi	
		R_r	ξ_r	R_r	ξ_r
.70	1.0	1.078	0.	1.075	0.
	.9	1.079	0.002	1.075	0.001
	.75	1.079	0.004	1.074	0.003
	.5	1.076	0.008	1.072	0.006
	.25	1.072	0.010	1.069	0.008
	.0	1.066	0.009	1.064	0.009
.65	1.0	1.053	0.	1.051	0.
	.9	1.053	0.001	1.050	0.001
	.75	1.052	0.003	1.050	0.002
	.5	1.050	0.005	1.049	0.004
	.25	1.047	0.007	1.047	0.005
	.0	1.044	0.007	1.044	0.006
.60	1.0	1.034	0.	1.033	0.
	.9	1.034	0.001	1.033	0.000
	.75	1.034	0.001	1.033	0.001
	.5	1.033	0.003	1.032	0.002
	.25	1.031	0.004	1.031	0.003
	.0	1.029	0.004	1.029	0.004
.55	1.0	1.021	0.	1.021	0.
	.9	1.022	0.000	1.021	0.000
	.75	1.021	0.001	1.021	0.001
	.5	1.021	0.002	1.021	0.001
	.25	1.020	0.002	1.020	0.002
	.0	1.019	0.003	1.019	0.002
.50	1.0	1.013	0.	1.013	0.
	.9	1.013	0.000	1.013	0.000
	.75	1.013	0.000	1.013	0.000
	.5	1.013	0.001	1.013	0.001
	.25	1.012	0.001	1.012	0.001
	.0	1.012	0.002	1.012	0.001

Table 4 -- Standard Values for R_f and ξ_f, the Period Lengthening Ratio and the Added Damping Ratio, due to Dam-Foundation Rock Interaction -- *Higher Dams*

E_f/E_s	$\eta_f = 0.01$		$\eta_f = 0.10$		$\eta_f = 0.25$		$\eta_f = 0.50$	
	R_f	ξ_f	R_f	ξ_f	R_f	ξ_f	R_f	ξ_f
5.0	1.058	0.017	1.055	0.020	1.050	0.026	1.040	0.032
4.5	1.064	0.019	1.060	0.022	1.055	0.029	1.044	0.036
4.0	1.071	0.022	1.067	0.025	1.061	0.032	1.048	0.040
3.5	1.081	0.025	1.076	0.029	1.068	0.037	1.054	0.045
3.0	1.093	0.028	1.087	0.033	1.078	0.042	1.062	0.052
2.5	1.110	0.034	1.102	0.040	1.092	0.050	1.072	0.061
2.0	1.134	0.041	1.124	0.049	1.111	0.062	1.087	0.075
1.5	1.172	0.053	1.159	0.064	1.142	0.080	1.110	0.097
1.4	1.182	0.057	1.169	0.068	1.150	0.085	1.116	0.103
1.3	1.194	0.060	1.180	0.072	1.160	0.090	1.123	0.109
1.2	1.207	0.065	1.192	0.078	1.171	0.096	1.131	0.117
1.1	1.221	0.069	1.207	0.084	1.183	0.104	1.140	0.126
1.0	1.240	0.075	1.224	0.091	1.198	0.112	1.151	0.136
0.9	1.261	0.082	1.244	0.099	1.215	0.122	1.163	0.149
0.8	1.287	0.090	1.269	0.109	1.236	0.134	1.178	0.163
0.7	1.319	0.100	1.299	0.120	1.262	0.149	1.196	0.182
0.6	1.362	0.112	1.338	0.135	1.295	0.167	1.219	0.205
0.5	1.417	0.128	1.389	0.154	1.339	0.190	1.249	0.235
0.4	1.491	0.150	1.462	0.178	1.399	0.221	1.289	0.276
0.3	1.612	0.181	1.572	0.212	1.490	0.265	1.347	0.338
0.2	1.829	0.218	1.764	0.261	1.646	0.333	1.438	0.443

Table 5 -- Standard Values for R_f and ξ_f, the Period Lengthening Ratio and the Added Damping Ratio, due to Dam-Foundation Rock Interaction -- *Lower Dams*

E_f/E_s	$\eta_f = 0.01$		$\eta_f = 0.10$		$\eta_f = 0.25$		$\eta_f = 0.50$	
	R_f	ξ_f	R_f	ξ_f	R_f	ξ_f	R_f	ξ_f
5.0	1.056	0.011	1.054	0.017	1.050	0.023	1.040	0.030
4.5	1.062	0.012	1.060	0.019	1.055	0.026	1.044	0.033
4.0	1.070	0.014	1.066	0.021	1.061	0.029	1.049	0.037
3.5	1.079	0.016	1.075	0.024	1.069	0.033	1.055	0.042
3.0	1.091	0.018	1.086	0.028	1.079	0.038	1.063	0.048
2.5	1.108	0.022	1.102	0.033	1.093	0.044	1.074	0.057
2.0	1.133	0.028	1.124	0.041	1.113	0.054	1.090	0.069
1.5	1.172	0.037	1.161	0.054	1.145	0.070	1.115	0.089
1.4	1.183	0.039	1.171	0.057	1.154	0.074	1.121	0.094
1.3	1.195	0.042	1.182	0.061	1.164	0.079	1.129	0.100
1.2	1.209	0.046	1.195	0.065	1.176	0.085	1.138	0.107
1.1	1.225	0.050	1.210	0.070	1.189	0.091	1.148	0.115
1.0	1.244	0.055	1.228	0.076	1.205	0.098	1.160	0.124
0.9	1.266	0.061	1.249	0.083	1.224	0.107	1.175	0.135
0.8	1.293	0.068	1.275	0.091	1.247	0.117	1.192	0.148
0.7	1.325	0.077	1.308	0.100	1.275	0.129	1.213	0.165
0.6	1.366	0.088	1.349	0.112	1.318	0.144	1.240	0.185
0.5	1.421	0.103	1.405	0.127	1.360	0.163	1.276	0.210
0.4	1.505	0.121	1.483	0.146	1.429	0.188	1.327	0.245
0.3	1.643	0.143	1.604	0.172	1.534	0.223	1.402	0.295
0.2	1.870	0.166	1.818	0.208	1.719	0.273	1.533	0.374

Table 6(a) -- Standard Values for the Hydrodynamic Pressure Function $p(\hat{y})$ for Full Reservoir, i.e., $H/H_s = 1$; $\alpha = 1.00$ -- *Higher and Lower Dams*

$\hat{y}=y/H$	Value of $gp(\hat{y})/wH$						
	$R_w \le 0.5$	$R_w=0.7$	$R_w=0.8$	$R_w=0.85$	$R_w=0.9$	$R_w=0.92$	$R_w=0.93$
1.00	0	0	0	0	0	0	0
.95	.080	.083	.087	.090	.096	.099	.102
.90	.131	.138	.146	.152	.163	.170	.175
.85	.153	.163	.175	.184	.201	.211	.218
.80	.164	.178	.193	.206	.228	.242	.251
.75	.176	.194	.212	.228	.254	.271	.283
.70	.184	.204	.226	.244	.276	.296	.310
.65	.183	.207	.231	.253	.289	.312	.328
.60	.180	.206	.234	.258	.298	.325	.342
.55	.179	.207	.238	.264	.309	.338	.357
.50	.177	.207	.240	.269	.317	.349	.369
.45	.170	.203	.238	.269	.321	.355	.377
.40	.164	.198	.236	.268	.323	.359	.383
.35	.159	.196	.235	.269	.327	.365	.390
.30	.155	.193	.234	.269	.330	.369	.395
.25	.149	.188	.230	.267	.330	.370	.397
.20	.143	.183	.226	.264	.329	.371	.399
.15	.141	.181	.225	.264	.330	.373	.401
.10	.139	.179	.224	.263	.330	.374	.403
.05	.135	.176	.222	.261	.329	.373	.402
0	.133	.175	.220	.260	.327	.372	.401

Table 6(a) -- Continued

$\hat{y}=y/H$	Value of $gp(\hat{y})/wH$					
	$R_w=0.94$	$R_w=0.95$	$R_w=0.96$	$R_w=0.97$	$R_w=0.98$	$R_w=0.99$
1.00	0	0	0	0	0	0
.95	.105	.108	.113	.121	.133	.161
.90	.181	.188	.198	.213	.237	.293
.85	.227	.238	.253	.275	.311	.394
.80	.262	.276	.296	.328	.374	.484
.75	.297	.315	.339	.375	.435	.572
.70	.326	.347	.376	.419	.490	.652
.65	.347	.371	.405	.453	.536	.722
.60	.363	.391	.428	.483	.576	.785
.55	.380	.411	.452	.513	.615	.847
.50	.395	.428	.473	.539	.651	.902
.45	.405	.440	.489	.560	.679	.950
.40	.412	.450	.502	.577	.705	.992
.35	.421	.461	.515	.595	.729	1.032
.30	.428	.469	.526	.609	.749	1.066
.25	.431	.474	.533	.619	.764	1.093
.20	.433	.477	.538	.627	.776	1.115
.15	.436	.482	.543	.634	.787	1.133
.10	.438	.485	.547	.640	.795	1.146
.05	.438	.484	.548	.641	.798	1.152
0	.437	.484	.547	.641	.798	1.154

Table 6(b) -- Standard Values for the Hydrodynamic Pressure Function $p(\hat{y})$ for Full Reservoir, i.e., $H/H_s = 1$; $\alpha = 0.90$ -- *Higher and Lower Dams*

$\hat{y}=y/H$	Value of $gp(\hat{y})/wH$								
	$R_w \leq 0.5$	$R_w=0.7$	$R_w=0.8$	$R_w=0.9$	$R_w=.95$	$R_w=1.0$	$R_w=1.05$	$R_w=1.1$	$R_w=1.2$
1.00	0	0	0	0	0	0	0	0	0
.95	.080	.083	.087	.095	.103	.104	.078	.071	.069
.90	.131	.138	.145	.161	.178	.178	.127	.114	.108
.85	.152	.163	.174	.198	.222	.223	.147	.127	.119
.80	.164	.178	.193	.223	.256	.257	.156	.130	.118
.75	.176	.193	.211	.249	.289	.290	.165	.133	.118
.70	.184	.204	.225	.270	.318	.318	.170	.131	.114
.65	.183	.206	.230	.282	.337	.337	.166	.122	.102
.60	.180	.206	.233	.290	.352	.352	.160	.109	.087
.55	.179	.207	.236	.300	.368	.368	.155	.099	.074
.50	.176	.207	.239	.308	.381	.381	.149	.088	.060
.45	.170	.202	.237	.311	.389	.388	.139	.074	.044
.40	.164	.198	.234	.312	.396	.394	.129	.059	.027
.35	.159	.195	.233	.315	.403	.400	.121	.047	.013
.30	.155	.192	.231	.317	.409	.405	.113	.036	.001
.25	.149	.187	.228	.316	.411	.407	.104	.024	.000
.20	.143	.182	.224	.315	.413	.407	.095	.013	.000
.15	.140	.180	.223	.316	.415	.408	.089	.005	.000
.10	.138	.179	.222	.316	.416	.408	.084	.000	.000
.05	.135	.176	.219	.314	.415	.406	.078	.000	.000
0	.133	.174	.217	.312	.414	.403	.074	.000	.000

Table 6(c) -- Standard Values for the Hydrodynamic Pressure Function $p(\hat{y})$ for Full Reservoir, i.e., $H/H_s = 1$; $\alpha = 0.75$ -- *Higher and Lower Dams*

$\hat{y}=y/H$	Value of $gp(\hat{y})/wH$								
	$R_w \leq 0.5$	$R_w=0.7$	$R_w=0.8$	$R_w=0.9$	$R_w=.95$	$R_w=1.0$	$R_w=1.05$	$R_w=1.1$	$R_w=1.2$
1.00	0	0	0	0	0	0	0	0	0
.95	.080	.083	.086	.091	.092	.090	.083	.077	.072
.90	.131	.137	.143	.153	.156	.151	.137	.126	.116
.85	.152	.162	.171	.185	.190	.182	.161	.145	.130
.80	.164	.177	.189	.207	.213	.203	.175	.153	.133
.75	.176	.192	.206	.228	.236	.223	.189	.161	.136
.70	.183	.202	.219	.245	.254	.239	.198	.165	.135
.65	.183	.204	.224	.253	.263	.245	.198	.160	.126
.60	.180	.203	.225	.258	.269	.249	.196	.153	.114
.55	.178	.204	.228	.264	.276	.253	.194	.147	.103
.50	.176	.204	.230	.268	.281	.256	.191	.139	.092
.45	.170	.199	.227	.268	.282	.254	.184	.128	.077
.40	.163	.194	.223	.267	.281	.250	.176	.116	.061
.35	.159	.191	.221	.267	.281	.249	.170	.106	.049
.30	.154	.188	.219	.266	.281	.246	.163	.097	.037
.25	.148	.183	.215	.263	.278	.241	.155	.086	.023
.20	.142	.178	.211	.260	.274	.235	.146	.075	.011
.15	.139	.175	.209	.258	.272	.232	.140	.068	.002
.10	.137	.173	.207	.257	.270	.229	.135	.061	.000
.05	.134	.170	.204	.254	.266	.223	.128	.054	.000
0	.132	.168	.202	.251	.263	.219	.123	.048	.000

Table 6(d) -- Standard Values for the Hydrodynamic Pressure Function $\hat{p}(\hat{y})$ for Full Reservoir, i.e., $H/H_s = 1$; $\alpha = 0.50$ -- *Higher and Lower Dams*

$\hat{y}=y/H$	Value of $g\hat{p}(\hat{y})/wH$								
	$R_w \le 0.5$	$R_w=0.7$	$R_w=0.8$	$R_w=0.9$	$R_w=.95$	$R_w=1.0$	$R_w=1.05$	$R_w=1.1$	$R_w=1.2$
1.00	0	0	0	0	0	0	0	0	0
.95	.079	.082	.083	.084	.084	.083	.082	.080	.077
.90	.130	.135	.138	.140	.139	.137	.134	.131	.125
.85	.151	.158	.163	.166	.165	.162	.158	.152	.143
.80	.163	.172	.178	.181	.180	.176	.170	.163	.151
.75	.174	.185	.192	.196	.195	.190	.183	.174	.158
.70	.181	.194	.202	.207	.205	.199	.190	.180	.161
.65	.181	.195	.204	.209	.207	.200	.189	.177	.155
.60	.177	.193	.203	.208	.206	.198	.185	.171	.146
.55	.176	.193	.204	.209	.206	.197	.183	.167	.138
.50	.173	.191	.202	.208	.204	.194	.178	.160	.129
.45	.166	.186	.197	.202	.198	.186	.170	.150	.116
.40	.159	.180	.191	.196	.191	.178	.160	.139	.103
.35	.155	.175	.187	.192	.186	.172	.152	.130	.091
.30	.150	.171	.183	.187	.180	.166	.145	.121	.080
.25	.144	.165	.177	.180	.172	.157	.134	.109	.066
.20	.138	.159	.171	.173	.165	.148	.124	.098	.054
.15	.134	.156	.167	.168	.159	.141	.117	.090	.044
.10	.132	.153	.164	.164	.154	.135	.110	.082	.035
.05	.128	.149	.159	.158	.148	.128	.101	.073	.025
0	.126	.146	.156	.153	.142	.121	.094	.065	.017

Table 6(e) -- Standard Values for the Hydrodynamic Pressure Function $p(\hat{y})$ for Full Reservoir, i.e., $H/H_s = 1$; $\alpha = 0.25$ -- *Higher and Lower Dams*

$\hat{y}=y/H$	Value of $gp(\hat{y})/wH$								
	$R_w \leq 0.5$	$R_w=0.7$	$R_w=0.8$	$R_w=0.9$	$R_w=.95$	$R_w=1.0$	$R_w=1.05$	$R_w=1.1$	$R_w=1.2$
1.00	0	0	0	0	0	0	0	0	0
.95	.079	.080	.080	.081	.081	.080	.080	.080	.079
.90	.129	.131	.132	.132	.132	.132	.131	.130	.129
.85	.150	.153	.154	.155	.154	.154	.153	.152	.149
.80	.160	.164	.166	.167	.166	.165	.164	.162	.158
.75	.171	.176	.178	.178	.178	.177	.175	.173	.168
.70	.178	.183	.185	.186	.185	.183	.181	.178	.172
.65	.176	.182	.184	.184	.183	.181	.178	.175	.168
.60	.172	.179	.181	.180	.179	.176	.173	.169	.160
.55	.170	.177	.178	.177	.175	.172	.169	.164	.153
.50	.166	.173	.175	.173	.171	.167	.163	.157	.145
.45	.159	.166	.167	.165	.162	.158	.152	.146	.133
.40	.152	.158	.159	.156	.152	.147	.141	.135	.120
.35	.146	.152	.152	.148	.144	.139	.132	.125	.108
.30	.141	.147	.146	.141	.137	.130	.123	.115	.097
.25	.134	.139	.138	.132	.126	.119	.111	.102	.083
.20	.127	.131	.129	.122	.116	.109	.100	.090	.069
.15	.124	.126	.124	.115	.109	.101	.091	.080	.059
.10	.120	.122	.118	.109	.102	.093	.083	.071	.048
.05	.116	.117	.112	.101	.093	.084	.073	.061	.037
0	.113	.112	.107	.095	.086	.076	.064	.052	.027

Table 6(f) -- Standard Values for the Hydrodynamic Pressure Function $p(\hat{y})$ for Full Reservoir, i.e., $H/H_s = 1$; $\alpha = 0.00$ -- *Higher and Lower Dams*

$\hat{y}=y/H$	Value of $gp(\hat{y})/wH$								
	$R_w \le 0.5$	$R_w=0.7$	$R_w=0.8$	$R_w=0.9$	$R_w=.95$	$R_w=1.0$	$R_w=1.05$	$R_w=1.1$	$R_w=1.2$
1.00	0	0	0	0	0	0	0	0	0
.95	.078	.078	.078	.079	.079	.079	.079	.079	.079
.90	.127	.127	.128	.128	.129	.129	.129	.129	.130
.85	.146	.147	.148	.149	.149	.149	.150	.150	.151
.80	.156	.157	.158	.158	.159	.159	.160	.160	.161
.75	.166	.167	.168	.168	.169	.169	.169	.170	.170
.70	.171	.172	.173	.173	.174	.174	.174	.175	.175
.65	.169	.170	.170	.170	.171	.171	.171	.171	.171
.60	.164	.164	.164	.164	.164	.164	.164	.164	.164
.55	.161	.160	.160	.160	.159	.159	.159	.158	.158
.50	.156	.155	.154	.153	.153	.152	.152	.151	.149
.45	.148	.146	.145	.143	.142	.142	.141	.139	.137
.40	.139	.136	.135	.132	.131	.130	.128	.127	.123
.35	.133	.129	.126	.123	.122	.120	.118	.116	.112
.30	.126	.121	.118	.114	.112	.110	.108	.105	.100
.25	.118	.112	.108	.103	.101	.098	.095	.092	.085
.20	.111	.103	.098	.092	.089	.086	.082	.079	.071
.15	.106	.096	.090	.084	.080	.076	.072	.068	.059
.10	.101	.090	.083	.076	.071	.067	.063	.058	.048
.05	.096	.083	.075	.066	.062	.057	.051	.046	.035
0	.092	.077	.068	.058	.053	.047	.042	.036	.023

Table 7(a) -- Standard Values for A_p, the Hydrodynamic Force Coefficient in $\tilde{L}_1$; $\alpha = 1$

R_w	Value of A_p for $\alpha = 1$
0.99	1.608
0.98	1.155
0.97	.955
0.96	.836
0.95	.755
0.94	.695
0.93	.649
0.92	.612
0.90	.555
0.85	.468
0.80	.417
0.70	.358
$\leq$ 0.50	.303

Table 7(b) -- Standard Values for A_p, the Hydrodynamic Force Coefficient in $\tilde{L}_1$; α = 0.90,0.75,0.50,0.25 and 0

R_w	Value of A_p				
	$\alpha = 0.90$	$\alpha = 0.75$	$\alpha = 0.50$	$\alpha = 0.25$	$\alpha = 0$
1.20	.088	.139	.201	.225	.230
1.10	.138	.225	.259	.250	.236
1.05	.247	.319	.292	.261	.239
1.00	.664	.437	.322	.271	.242
0.95	.669	.485	.342	.279	.245
0.90	.537	.464	.351	.286	.247
0.80	.414	.397	.345	.292	.252
0.70	.357	.351	.327	.292	.256
$\leq$ 0.50	.303	.302	.296	.283	.262

Table 8 -- Standard Values for the Hydrodynamic Pressure Function $p_o(\hat{y})$

$\hat{y} = y/H_s$	$gp_o(y)/wH$
1.0	0.
.95	.137
.90	.224
.85	.301
.80	.362
.75	.418
.70	.465
.65	.509
.60	.546
.55	.580
.50	.610
.45	.637
.40	.659
.35	.680
.30	.696
.25	.711
.20	.722
.15	.731
.10	.737
.05	.741
0.	.742

Table 9 -- Pine Flat Dam Analysis Cases, Simplified Procedure Parameters, and Fundamental Mode Properties

Case	Foundation Rock	Water	Parameters				Fundamental Mode Properties		
							Vibration Period in seconds	Damping Ratio	$S_a(\tilde{T}_1,\tilde{\xi}_1)$ in g's
			R_r	R_f	ξ_r	ξ_f	$\tilde{T}_1$	$\tilde{\xi}_1$	
1	rigid	empty	1.0	1.0	0	0	0.266	0.050	0.677
2	flexible	full	1.319	1.0	0.046	0	0.351	0.084	0.542
3	rigid	empty	1.0	1.224	0	0.091	0.326	0.118	0.453
4	flexible	full	1.319	1.224	0.046	0.091	0.429	0.158	0.377

Table 10 -- Equivalent Lateral Earthquake Forces on Pine Flat Dam due to Earthquake Ground Motion Characterized by the Smooth Design Spectrum of Fig. 15, Scaled by a Factor of 0.25

y (ft)	w_s (k/ft)	ϕ	$w_s\phi$ (k/ft)	gp (k/ft)	gp_o (k/ft)	Lateral Forces, in kips per foot							
						Case 1		Case 2		Case 3		Case 4	
						f_1	f_{sc}	f_1	f_{sc}	f_1	f_{sc}	f_1	f_{sc}
400.	0.74	1.0	0.74	0	0.	1.45	-0.35	1.26	-0.69	0.97	-0.35	0.87	-0.69
389.38	0.74	0.95	0.70	0	0.	1.38	-0.33	1.20	-0.65	0.93	-0.33	0.83	-0.65
359.286	1.52	0.81	1.23	1.98	3.54	2.43	-0.52	5.48	-0.20	1.62	-0.52	3.81	-0.20
340.	1.52	0.73	1.10	3.11	5.61	2.17	-0.42	7.16	0.48	1.45	-0.42	4.98	0.48
335.	5.39	0.71	3.81	3.26	6.09	7.48	-1.42	12.0	-1.64	5.00	-1.42	8.36	-1.64
308.85	9.70	0.61	5.87	3.84	8.30	11.5	-1.84	16.5	-2.46	7.72	-1.84	11.5	-2.46
300.	10.90	0.57	6.24	3.99	8.96	12.3	-1.80	17.4	-2.42	8.20	-1.80	12.1	-2.42
280.	13.47	0.50	6.79	4.29	10.3	13.3	-1.56	18.9	-2.11	8.93	-1.56	13.1	-2.11
240.	18.62	0.38	7.09	4.51	12.5	13.9	-0.50	19.7	-0.64	9.33	-0.50	13.7	-0.64
200.	23.72	0.28	6.57	4.51	14.1	12.9	1.16	18.9	1.68	8.64	1.16	13.1	1.68
160.	28.91	0.19	5.55	4.27	15.5	10.9	3.20	16.7	4.51	7.30	3.20	11.6	4.51
120.	34.05	0.12	4.19	4.08	16.4	8.23	5.47	14.1	7.66	5.51	5.47	9.79	7.66
80.	39.20	0.07	2.74	3.75	17.1	5.39	7.81	11.1	10.8	3.61	7.81	7.69	10.8
40.	44.35	0.03	1.33	3.54	17.5	2.62	10.1	8.29	13.9	1.75	10.1	5.76	13.9
0.	49.49	0.	0.	3.30	17.6	0.	12.4	5.62	16.8	0.	12.4	3.91	16.8

Table 11 -- Vertical Bending Stresses (in psi) at upstream and downstream faces of Pine Flat Dam

Elevation y (ft)	Case 1			Case 2			Case 3			Case 4		
	r_1	r_{sc}	r_d	r_1	r_{sc}	r_d	r_1	r_{sc}	r_d	r_1	r_{sc}	r_d
308.85	112	-23	114	185	-25	186	75	-23	78	128	-25	131
300	122	-24	124	200	-27	202	81	-24	85	139	-27	142
280	148	-28	151	238	-31	240	99	-28	103	166	-31	169
240	195	-31	197	302	-36	304	130	-31	134	210	-36	213
200	231	-31	233	350	-36	351	154	-31	157	243	-36	246
160	255	-26	257	383	-30	384	171	-26	173	266	-30	268
120	271	-18	271	405	-20	405	181	-18	182	281	-20	282
80	278	-8	278	417	-6	417	186	-8	186	290	-6	290
40	279	5	279	422	12	422	187	5	187	294	12	294
0	276	20	278	422	32	423	185	20	186	293	32	295

FIGURES

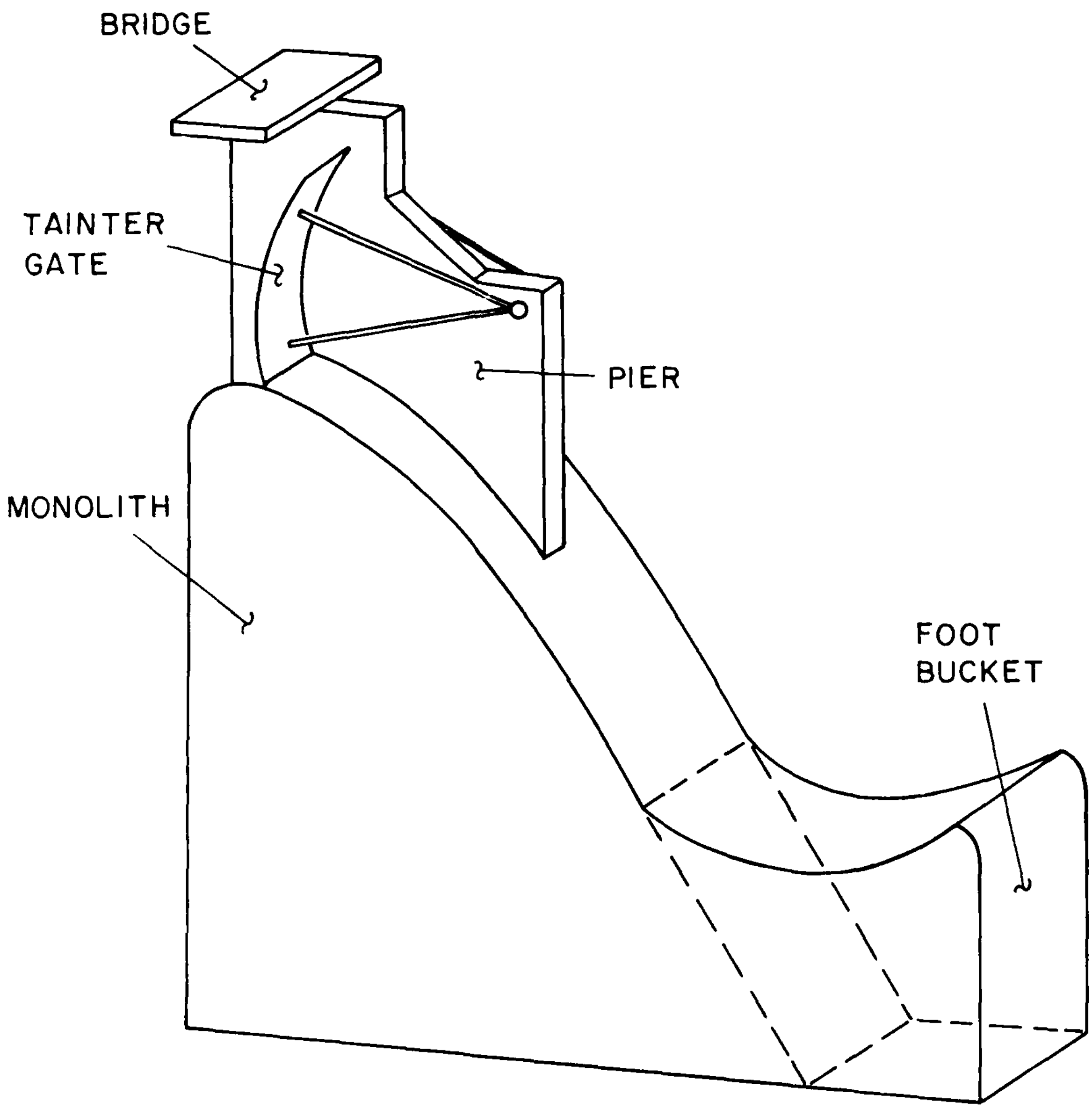

Figure 1 -- Gated Spillway Monolith

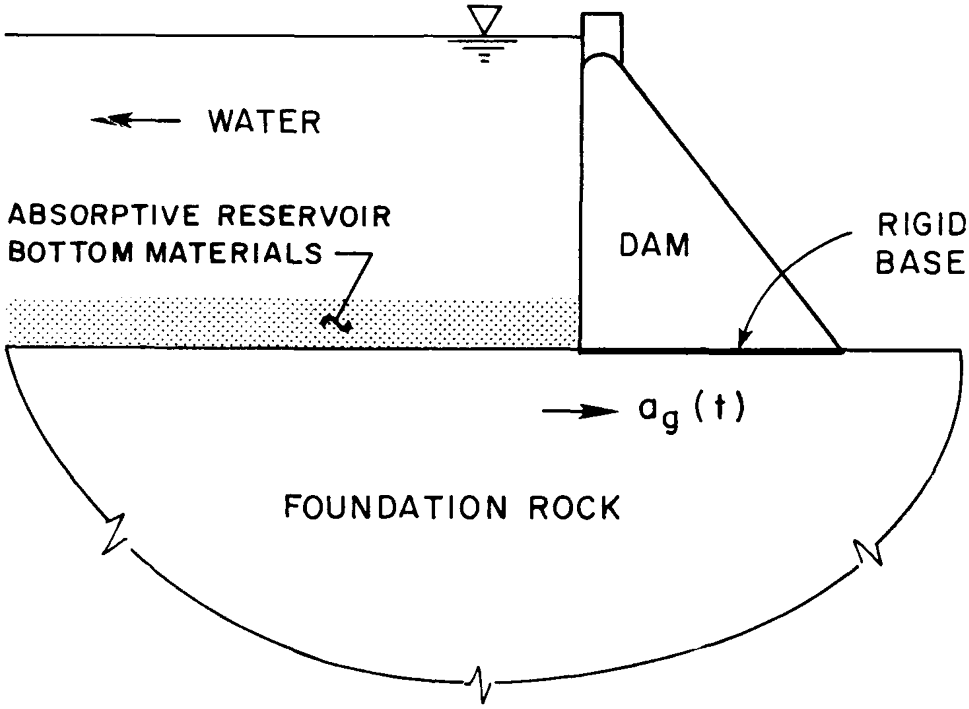

Figure 2 -- Dam-Water-Foundation System

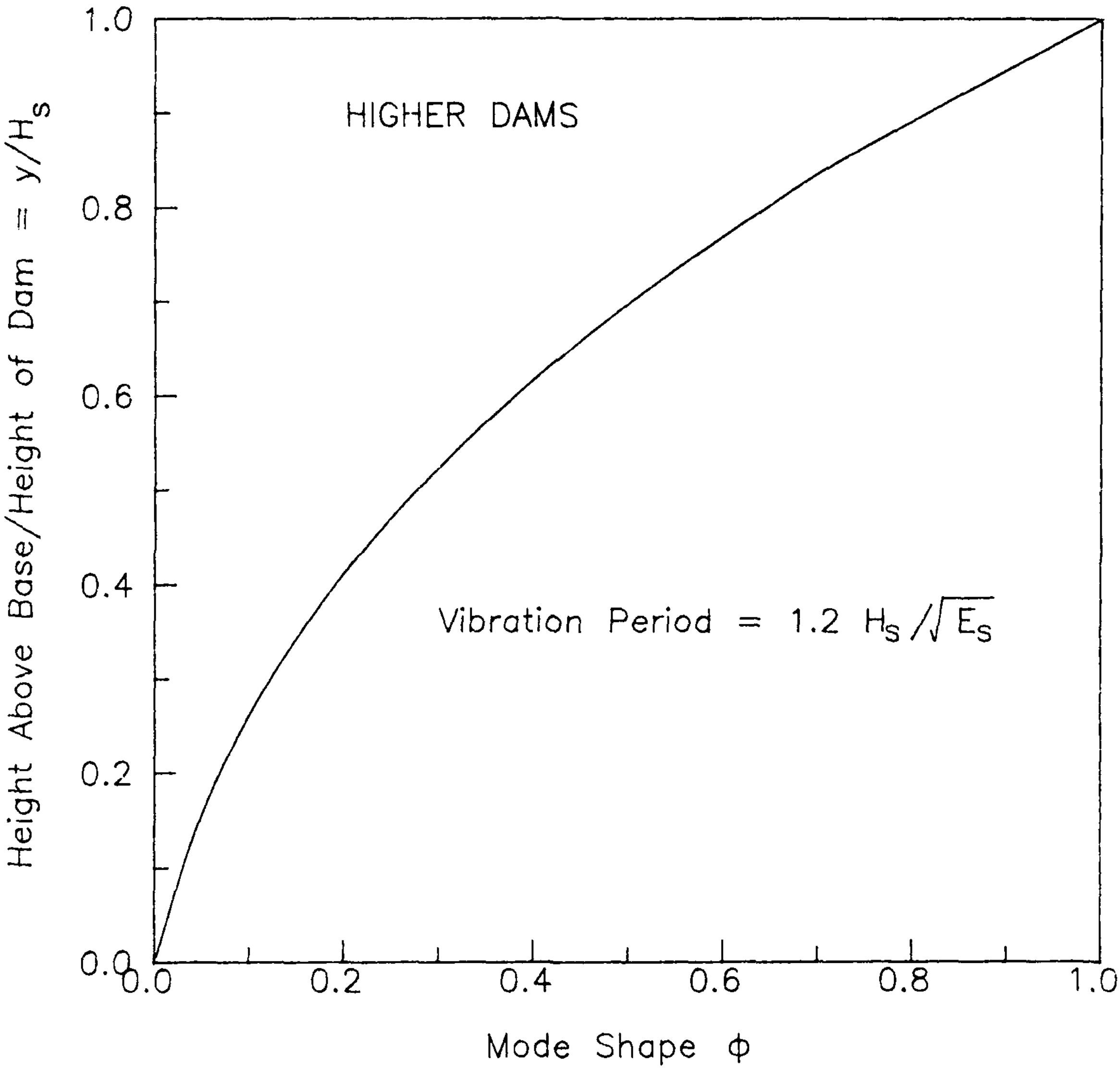

Figure 3(a) -- Standard Period and Mode Shape of Vibration for Gated Spillway Monoliths of Concrete Gravity Dams

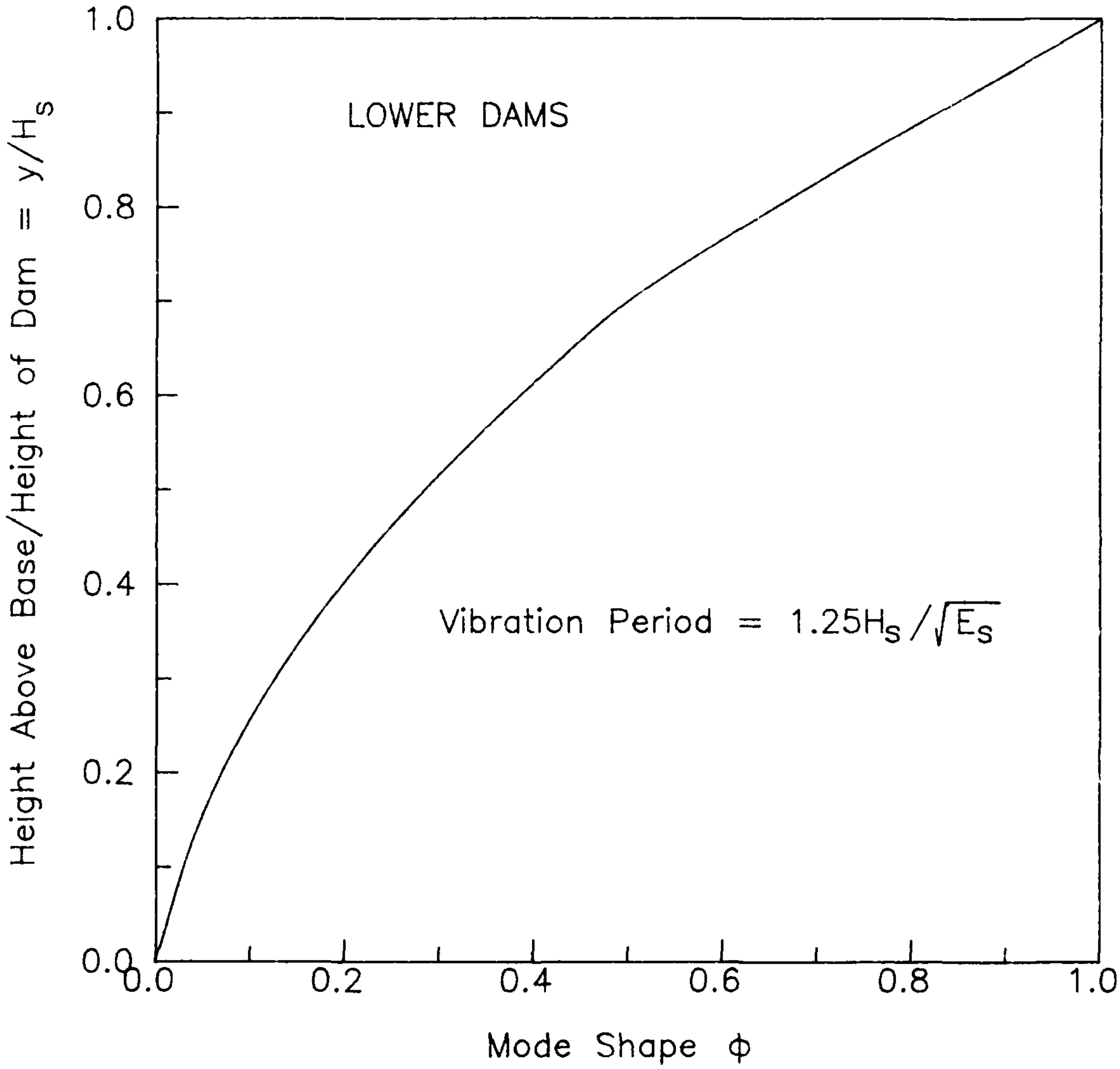

Figure 3(b) -- Standard Period and Mode Shape of Vibration for Gated Spillway Monoliths of Concrete Gravity Dams

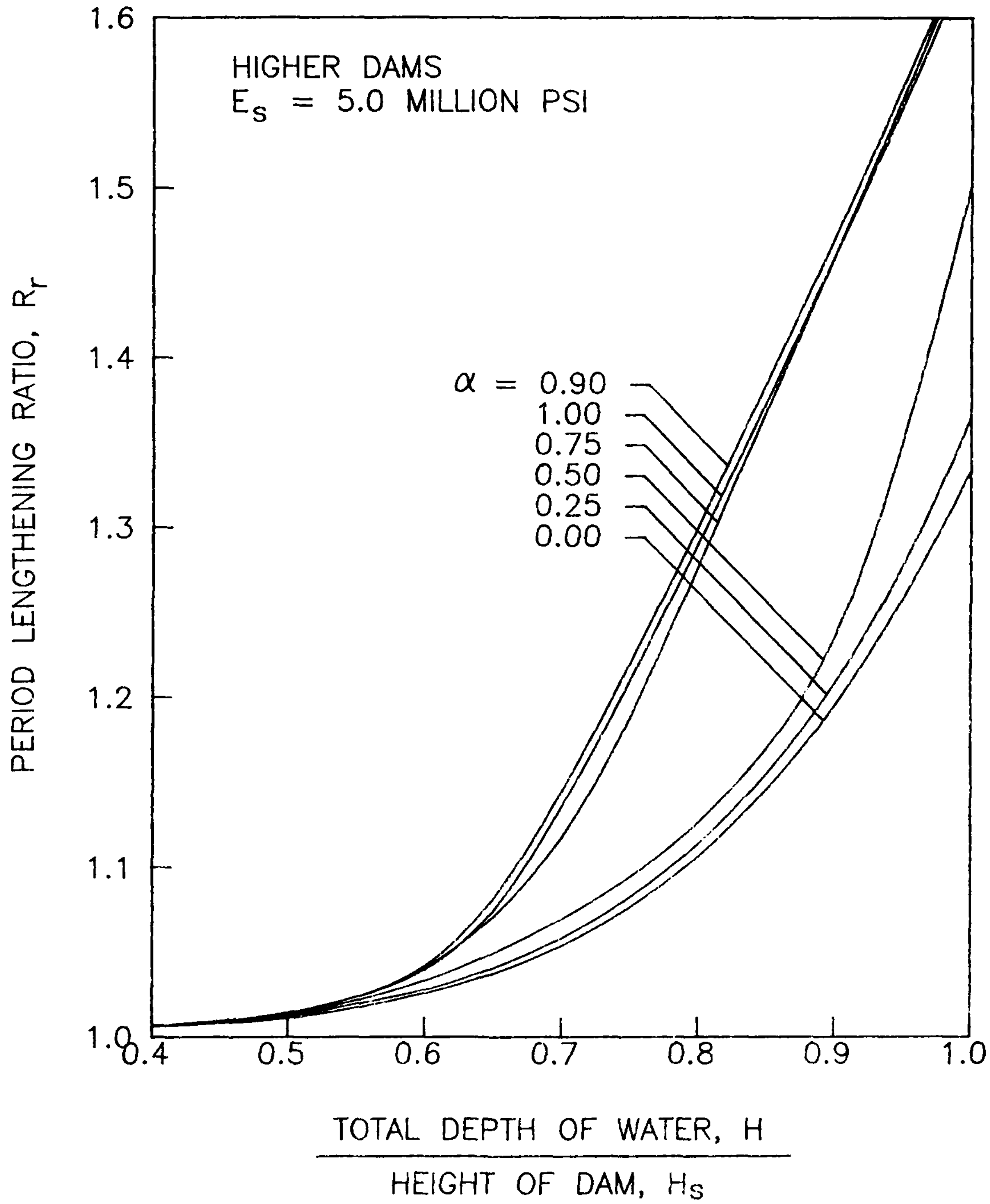

Figure 4(a) -- Standard Values for R_r, the Period Lengthening Ratio due to Hydrodynamic Effects; E_s = 5 million psi.

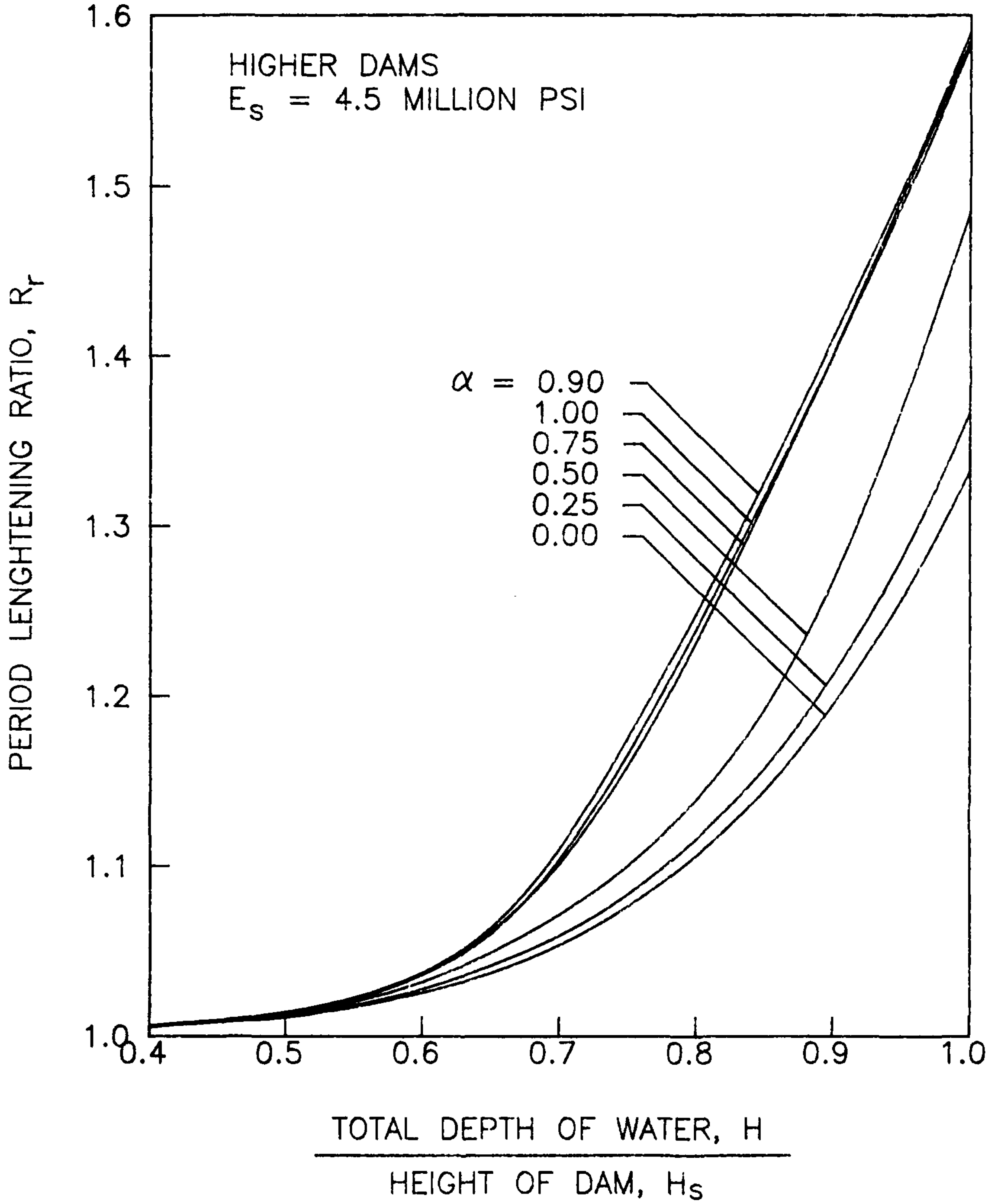

Figure 4(b) –– Standard Values for R_r, the Period Lengthening Ratio due to Hydrodynamic Effects; E_s = 4.5 million psi.

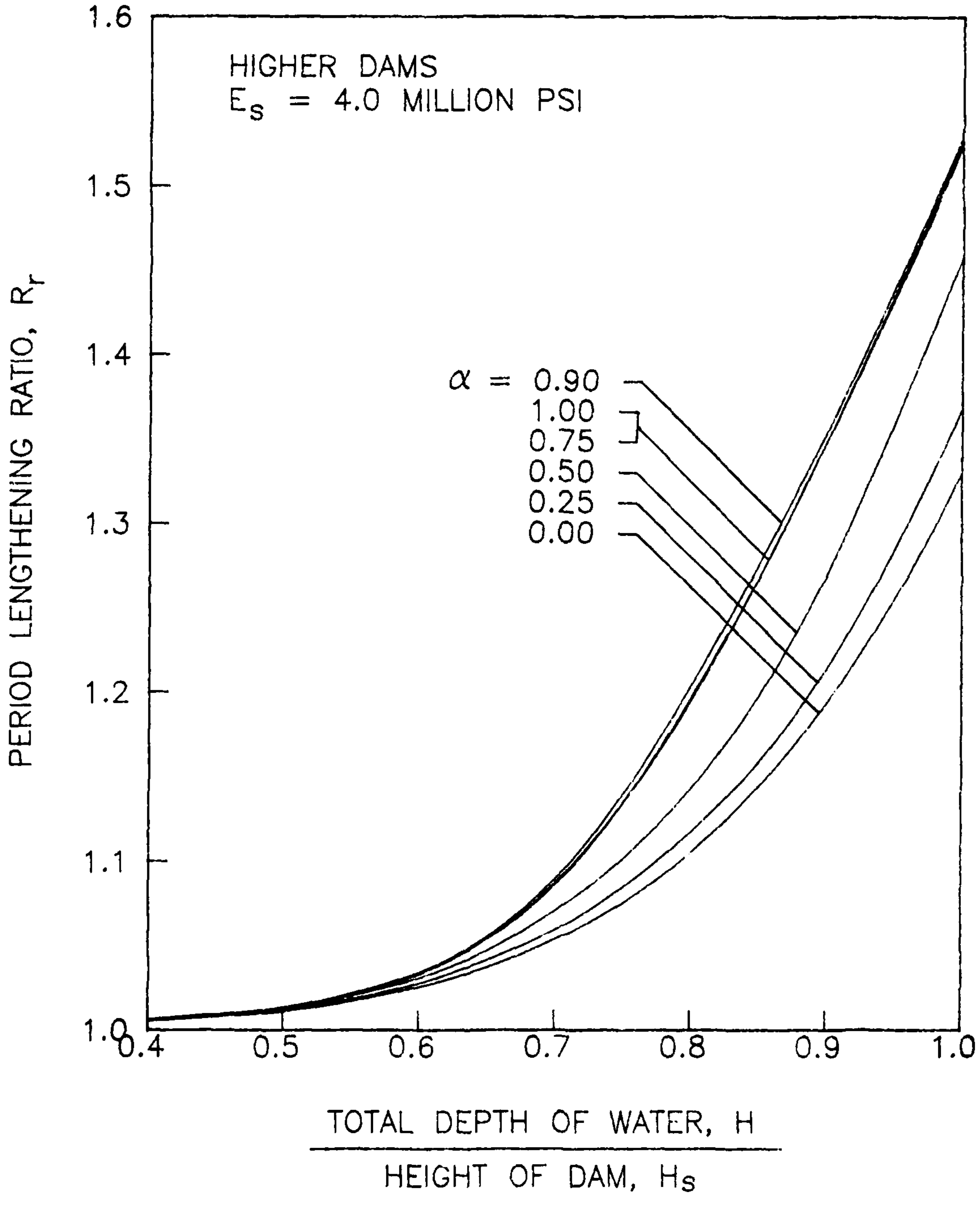

Figure 4(c) -- Standard Values for R_r, the Period Lengthening Ratio due to Hydrodynamic Effects; E_s = 4 million psi.

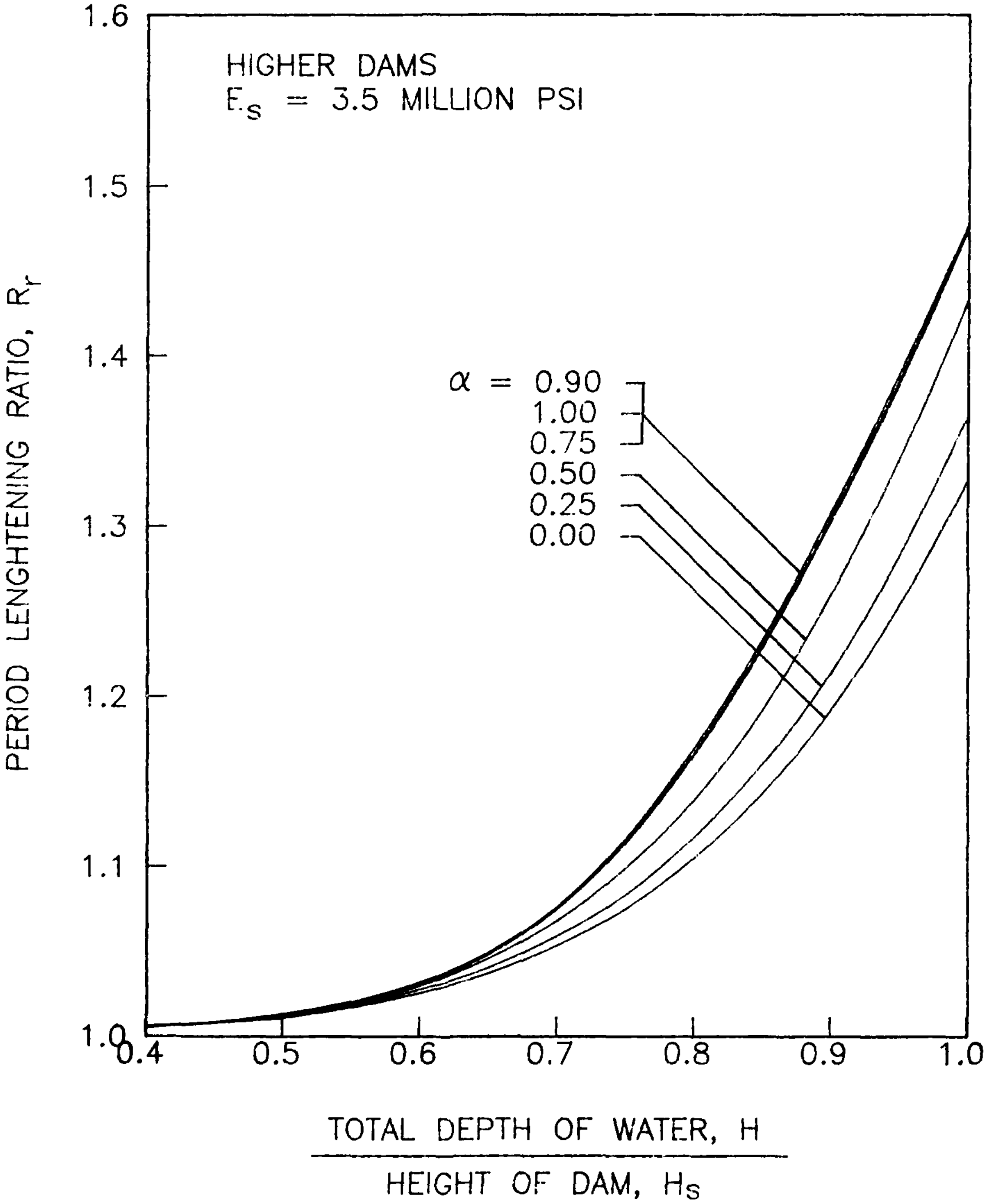

Figure 4(d) -- Standard Values for R_r, the Period Lengthening Ratio due to Hydrodynamic Effects; E_s = 3.5 million psi.

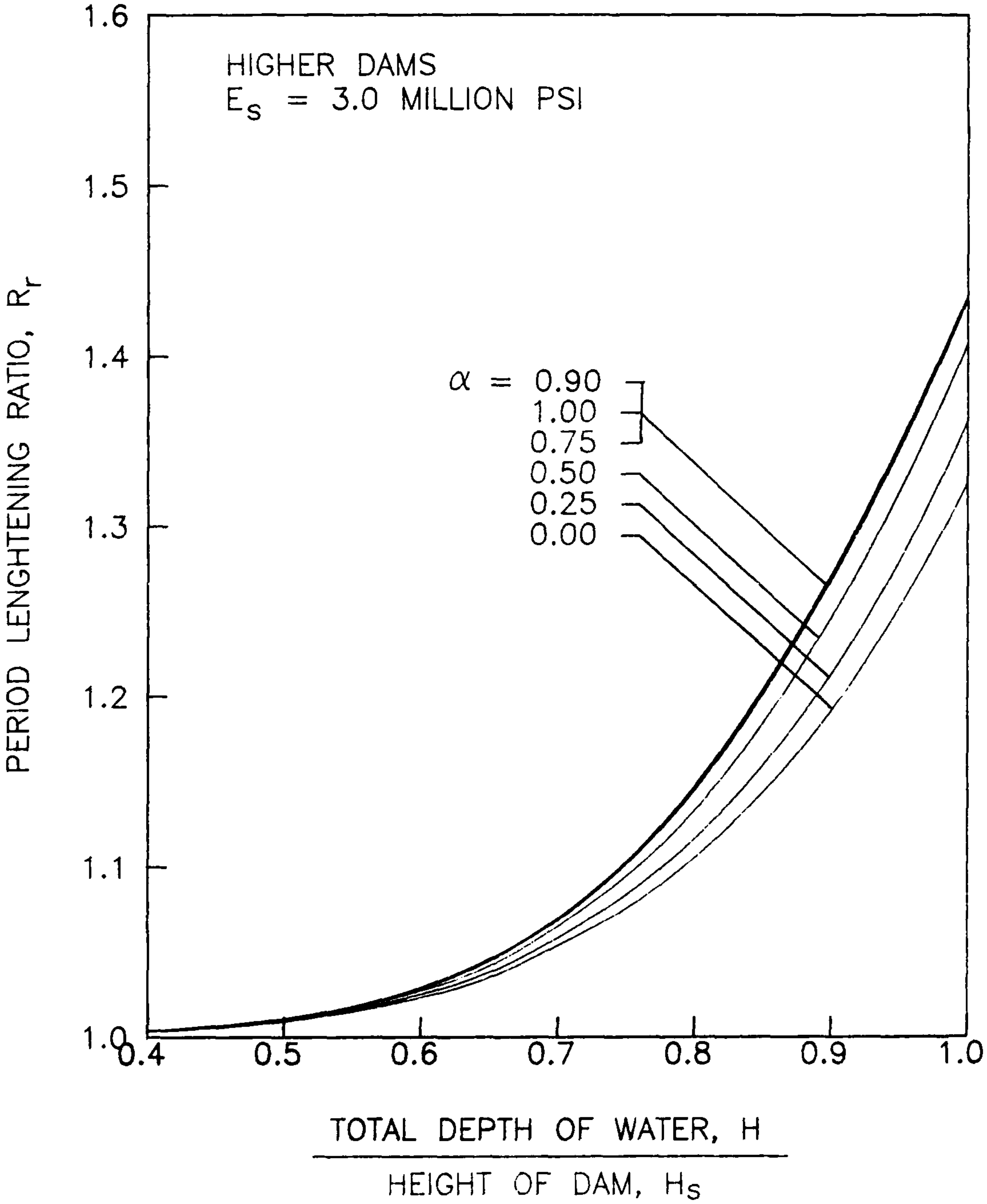

Figure 4(e) -- Standard Values for R_r, the Period Lengthening Ratio due to Hydrodynamic Effects; E_s = 3 million psi.

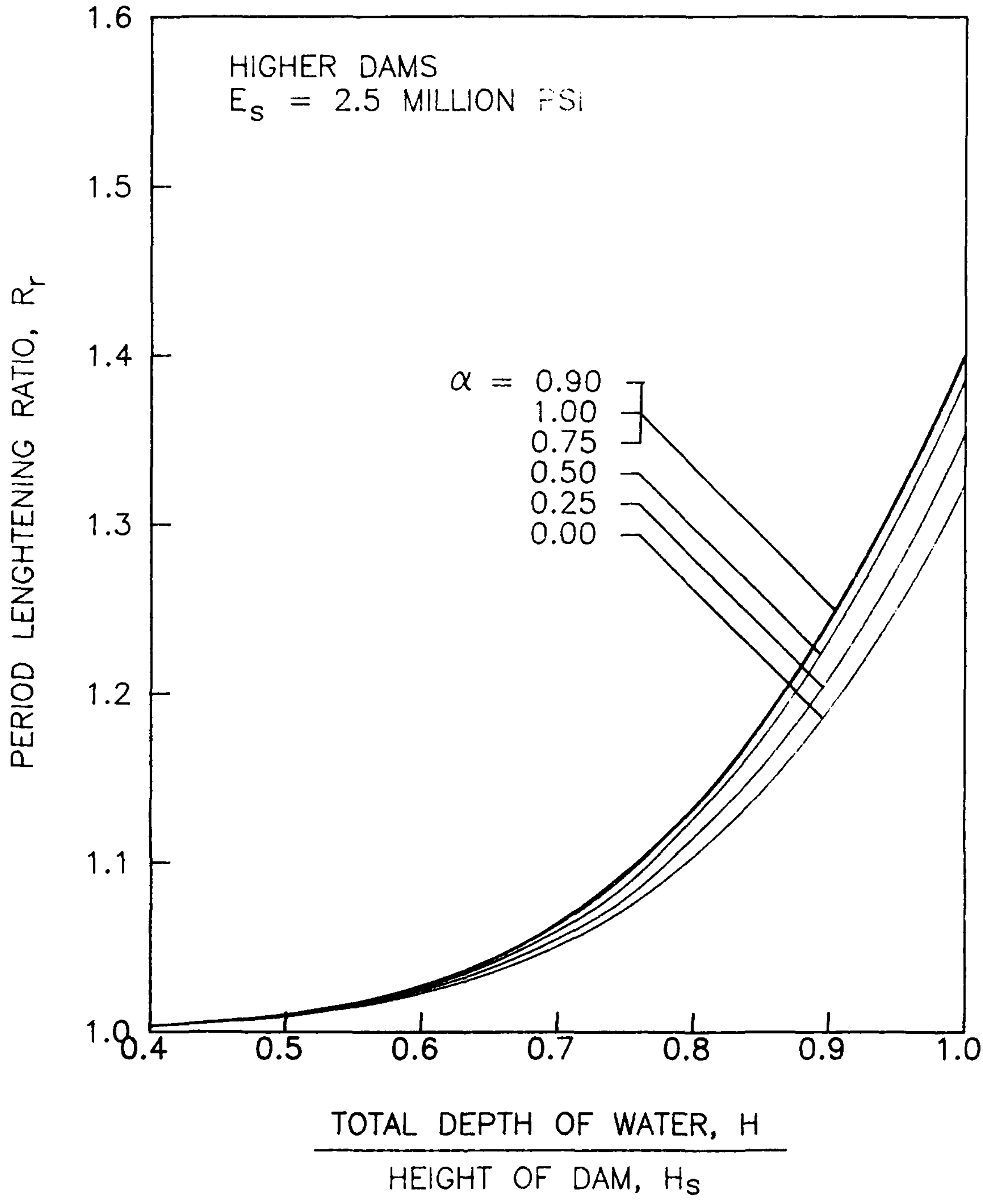

Figure 4(f) -- Standard Values for R_r, the Period Lengthening Ratio due to Hydrodynamic Effects; E_s = 2.5 million psi.

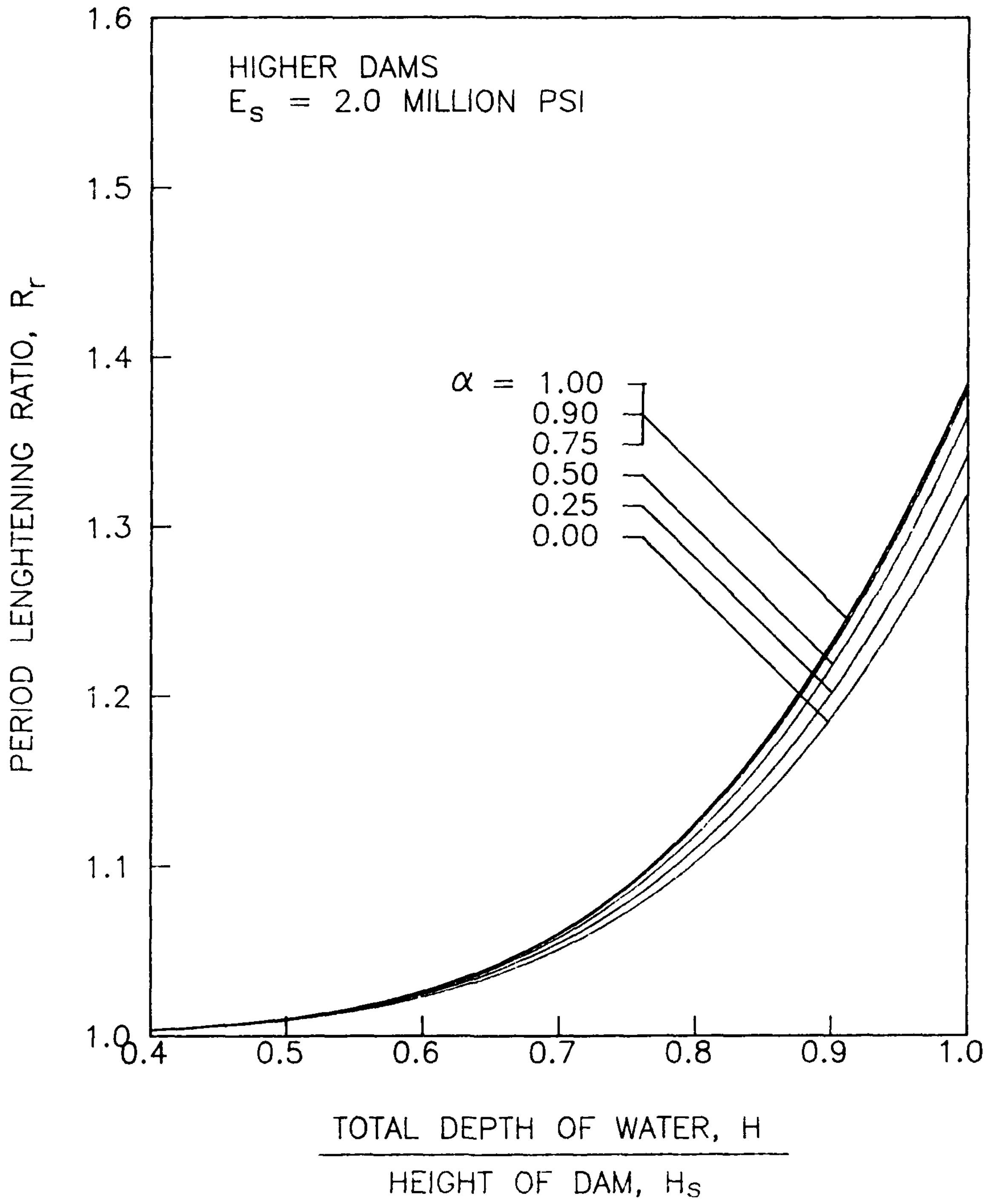

Figure 4(g) -- Standard Values for R_r, the Period Lengthening Ratio due to Hydrodynamic Effects; E_s = 2 million psi.

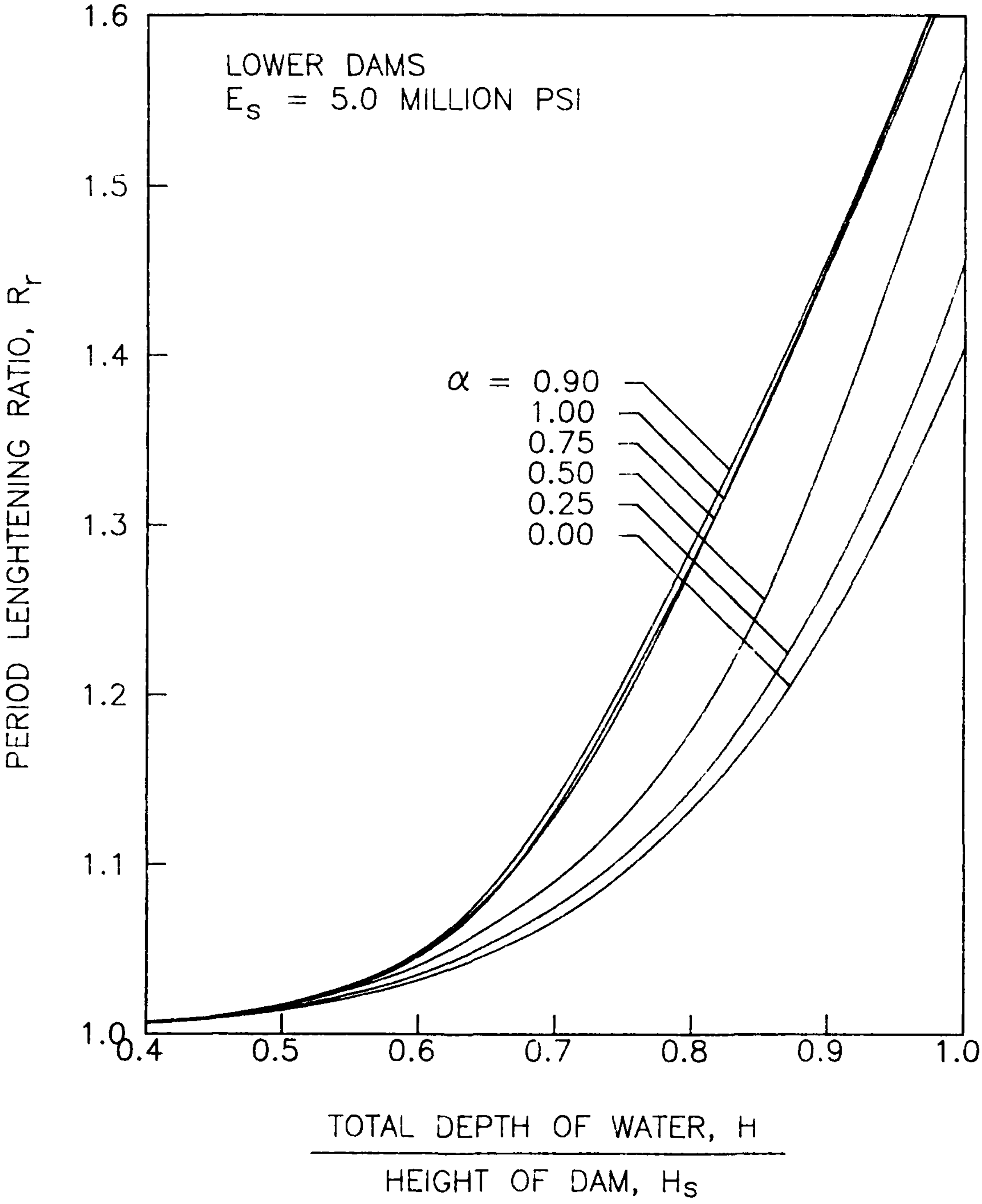

Figure 5(a) -- Standard Values for R_r, the Period Lengthening Ratio due to Hydrodynamic Effects; E_s = 5 million psi.

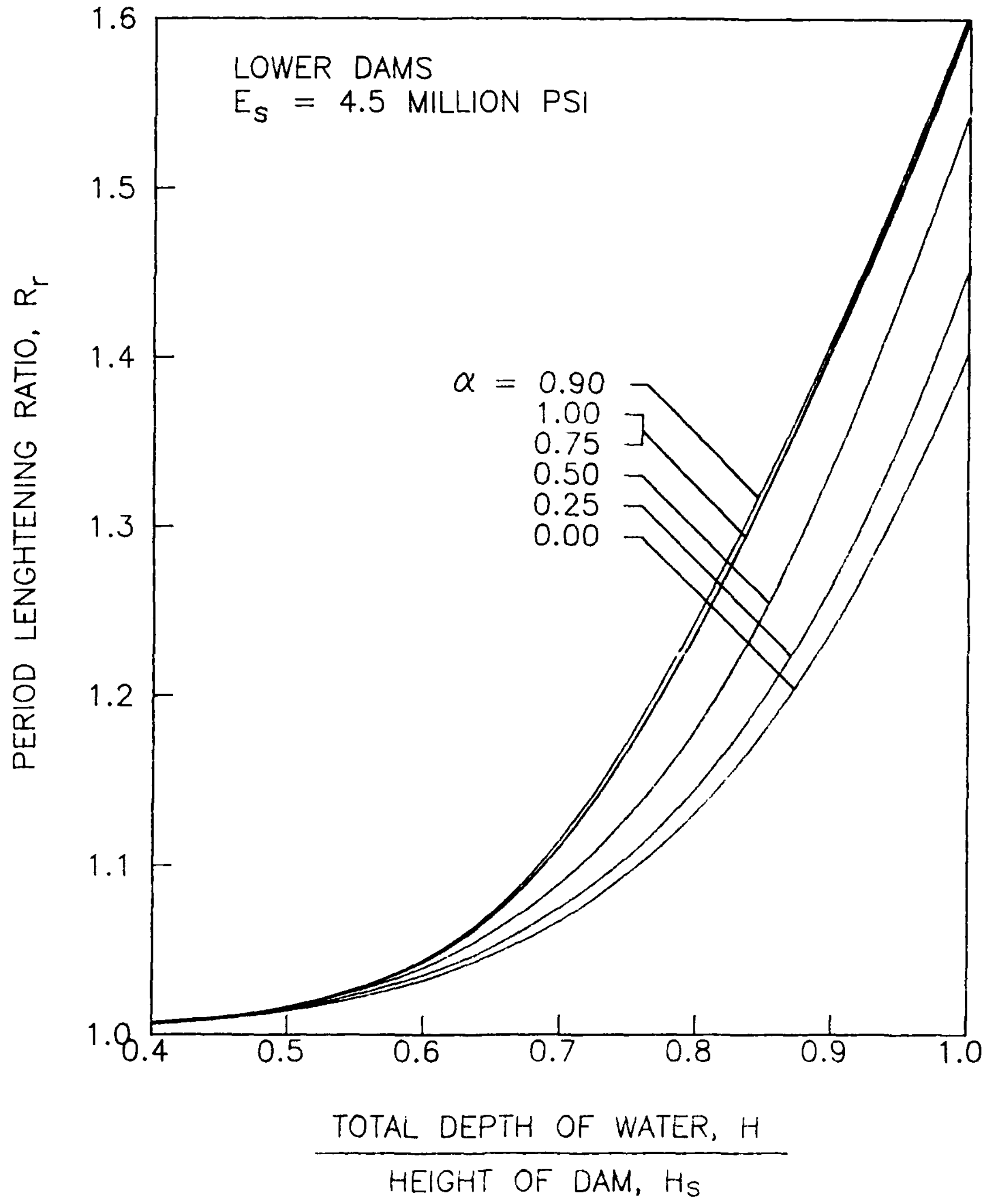

Figure 5(b) -- Standard Values for R_r, the Period Lengthening Ratio due to Hydrodynamic Effects; E_s = 4.5 million psi.

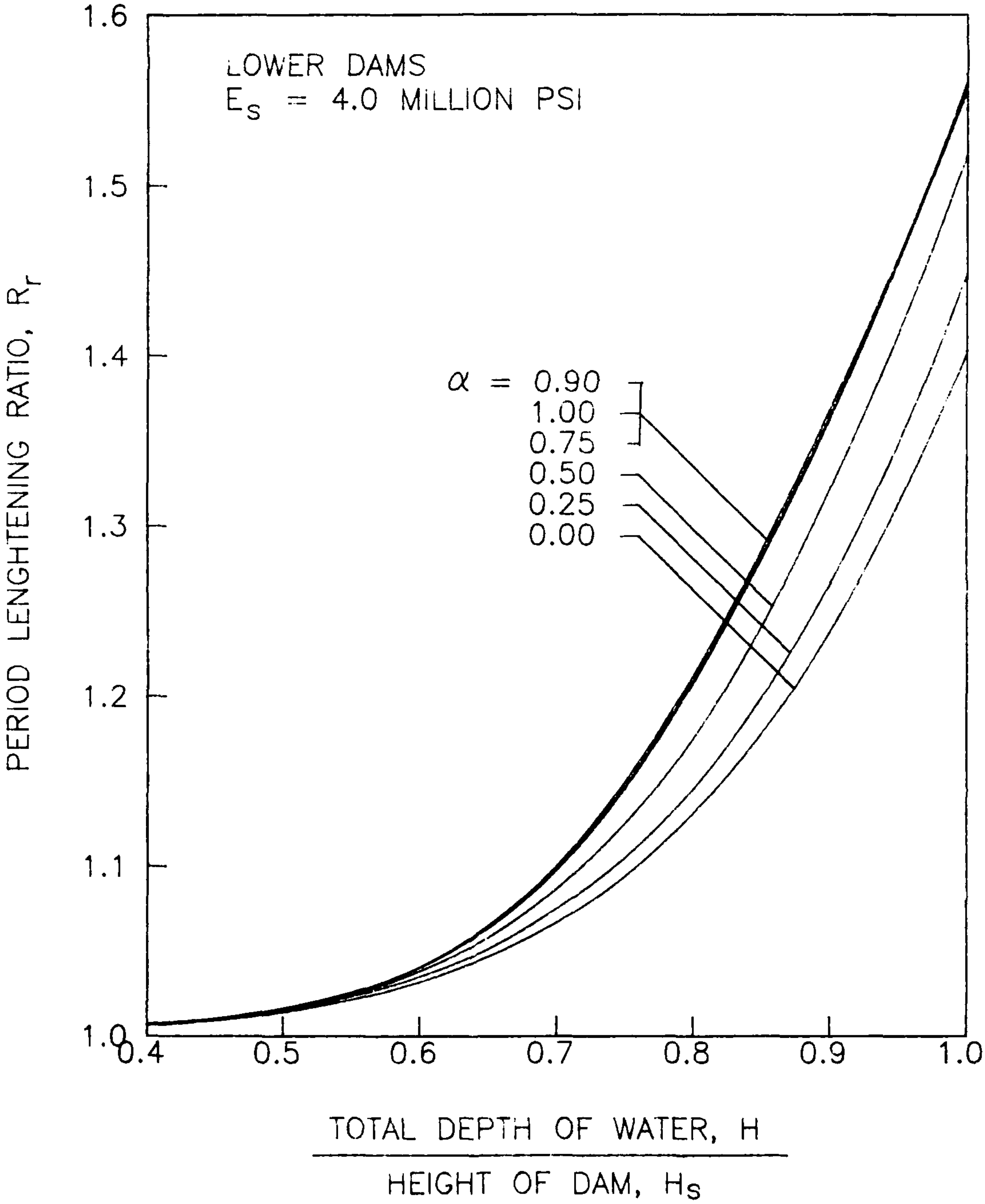

Figure 5(c) -- Standard Values for R_r, the Period Lengthening Ratio due to Hydrodynamic Effects; E_s = 4 million psi.

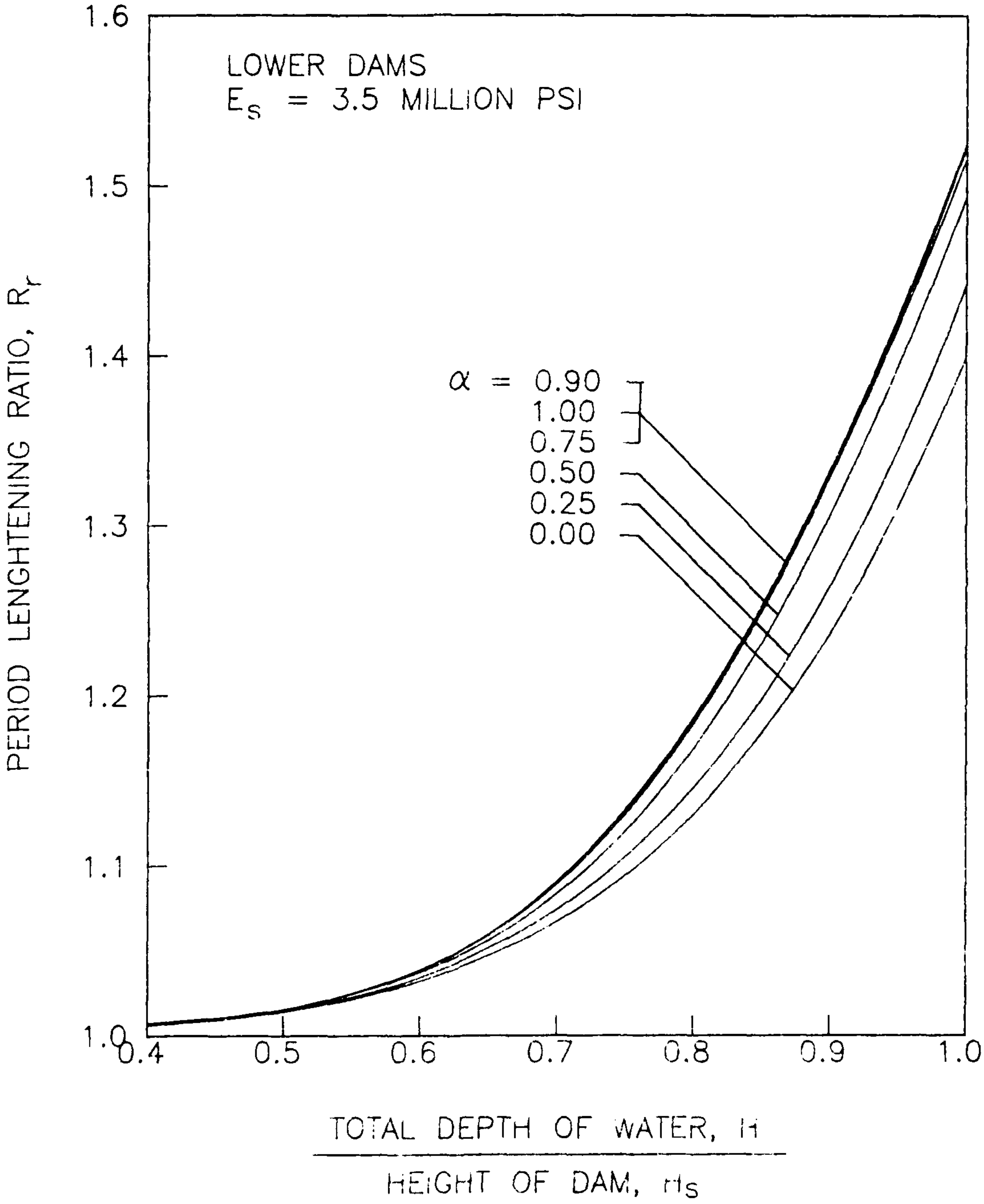

Figure 5(a) -- Standard Values for R_r, the Period Lengthening Ratio due to Hydrodynamic Effects; E_s = 3.5 million psi.

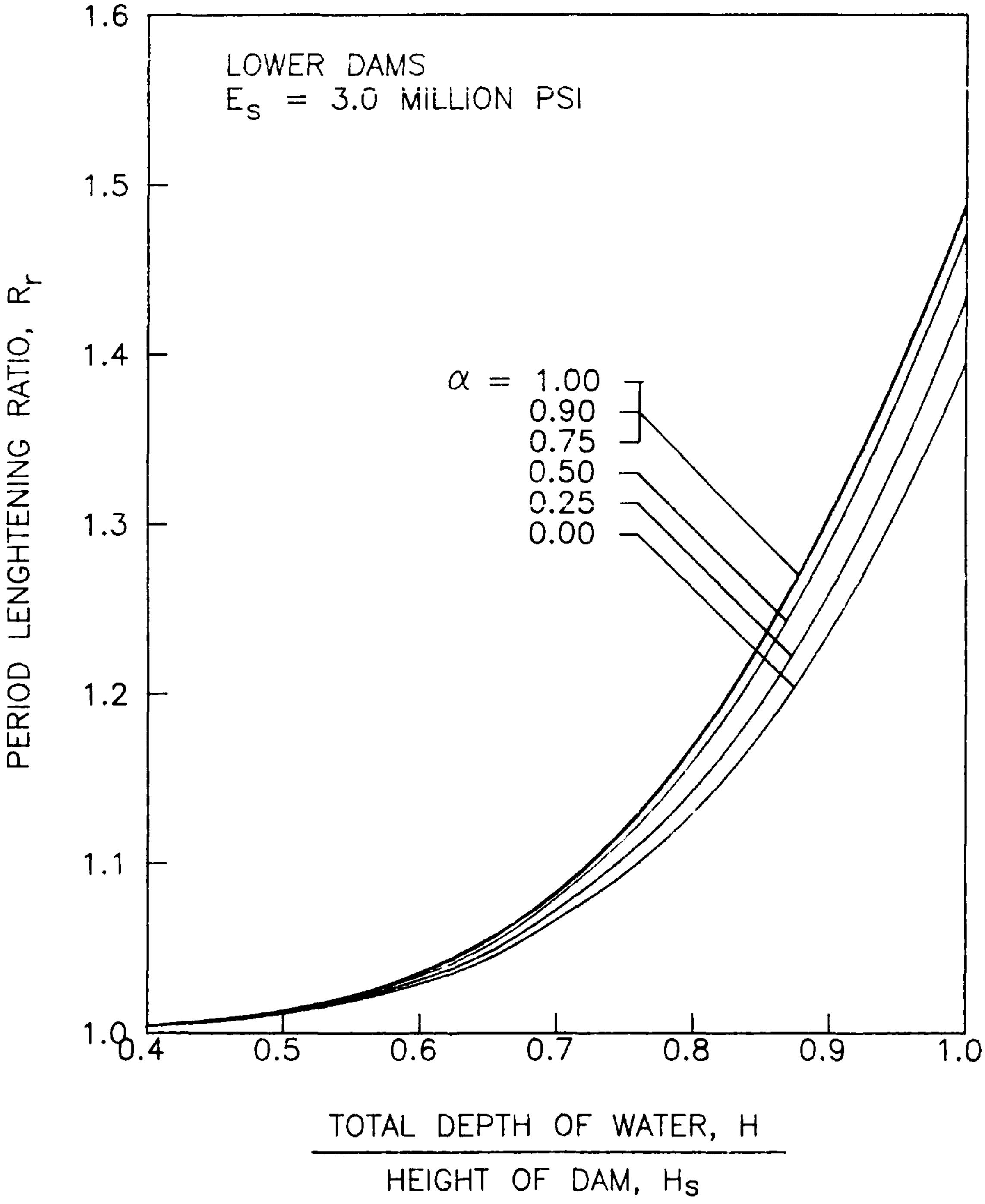

Figure 5(e) -- Standard Values for R_r, the Period Lengthening Ratio due to Hydrodynamic Effects; E_s = 3 million psi.

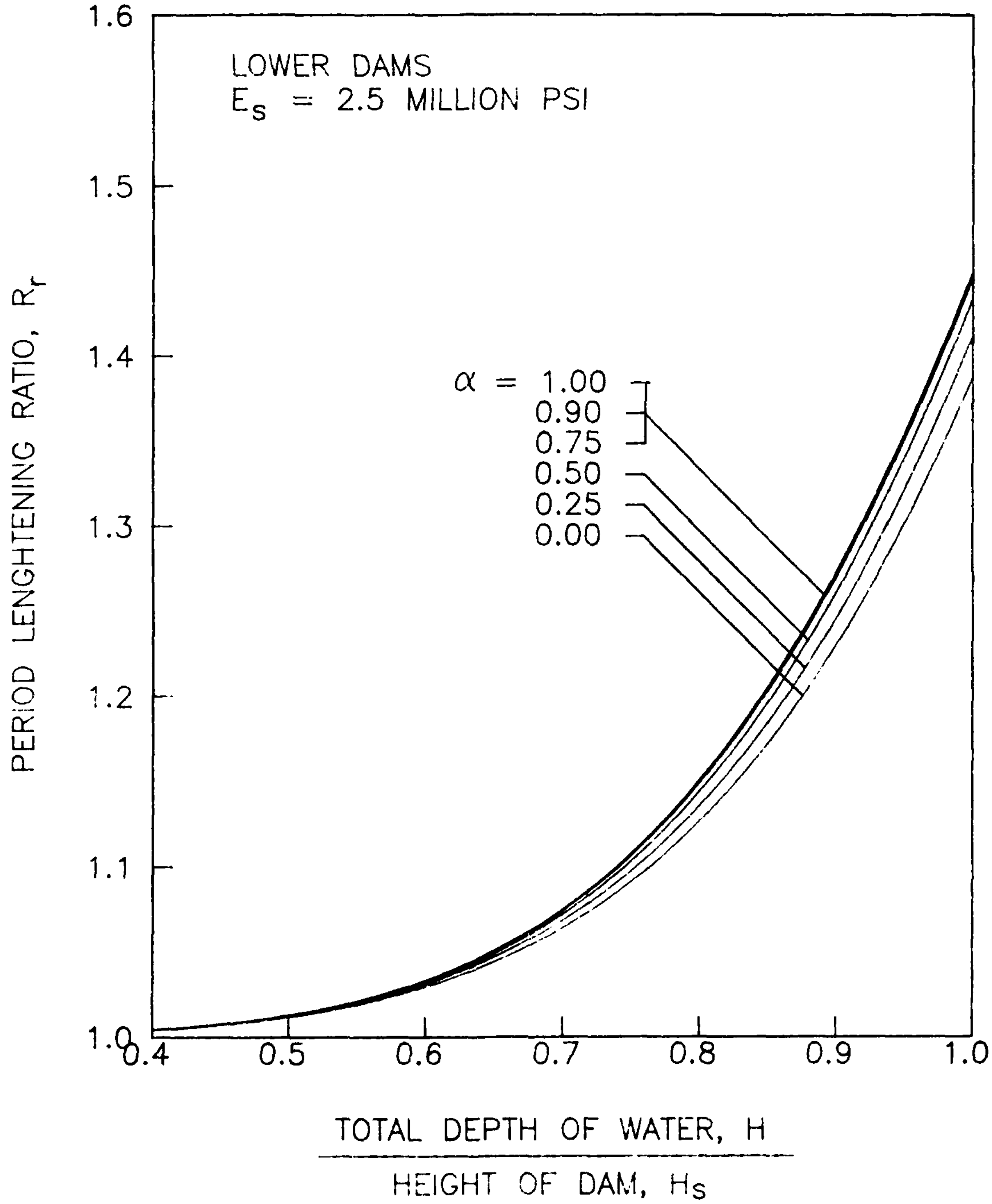

Figure 5(f) -- Standard Values for R_r, the Period Lengthening Ratio due to Hydrodynamic Effects; E_s = 2.5 million psi.

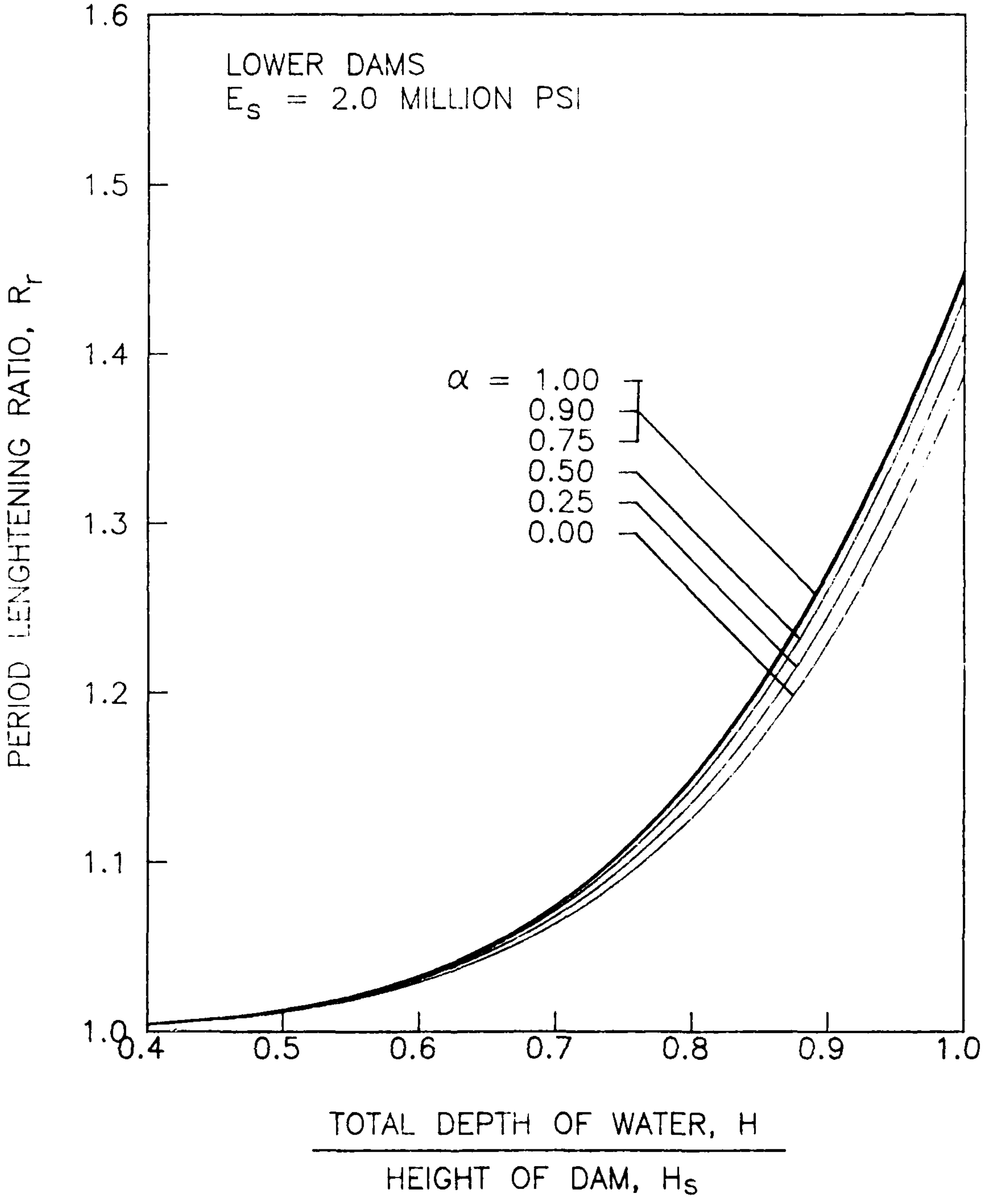

Figure 5(g) -- Standard Values for R_r, the Period Lengthening Ratio due to Hydrodynamic Effects; E_s = 2 million psi.

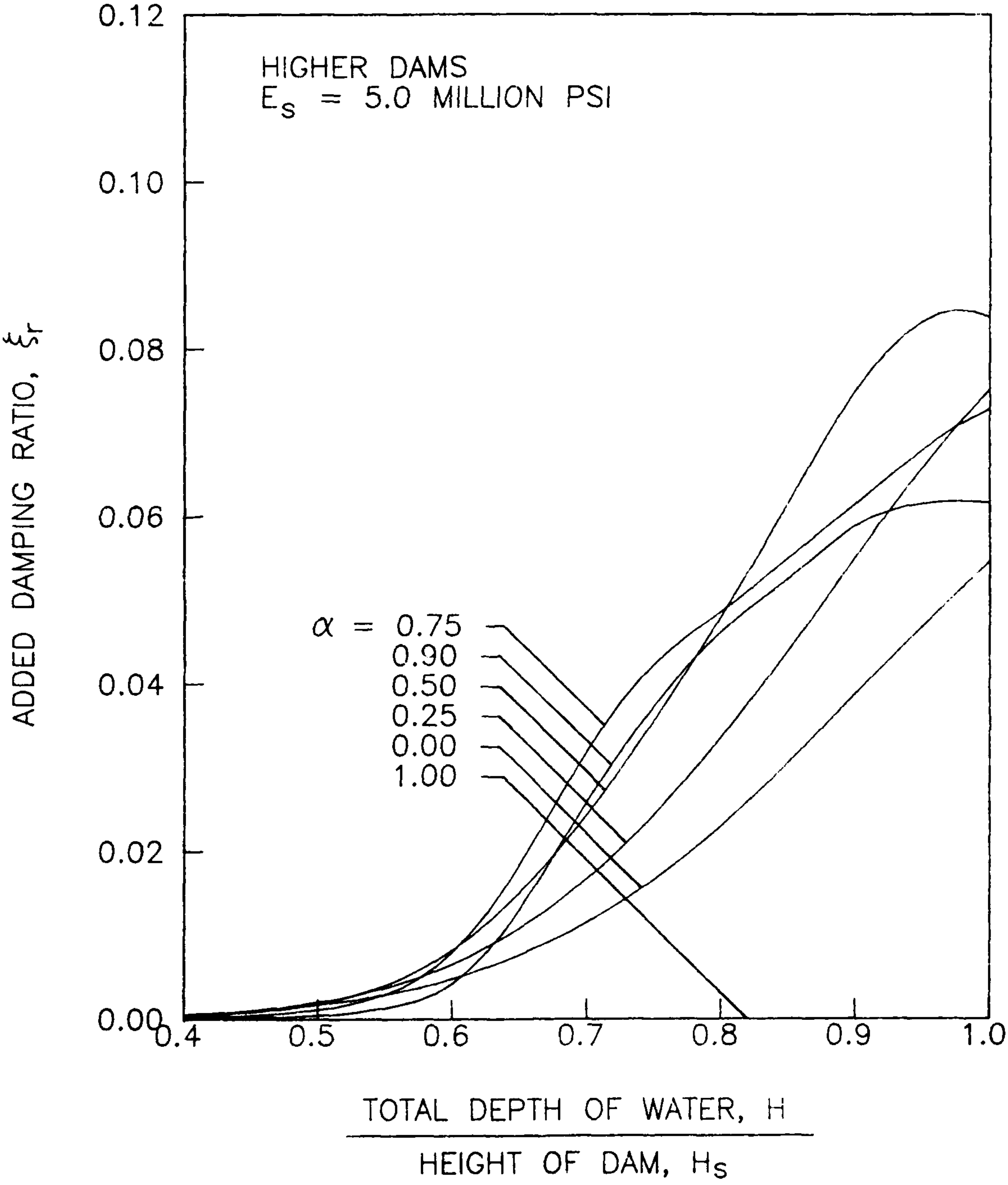

Figure 6(a) -- Standard Values for ξ_r, the Added Damping Ratio due to Hydrodynamic Effects; E_s = 5 million psi.

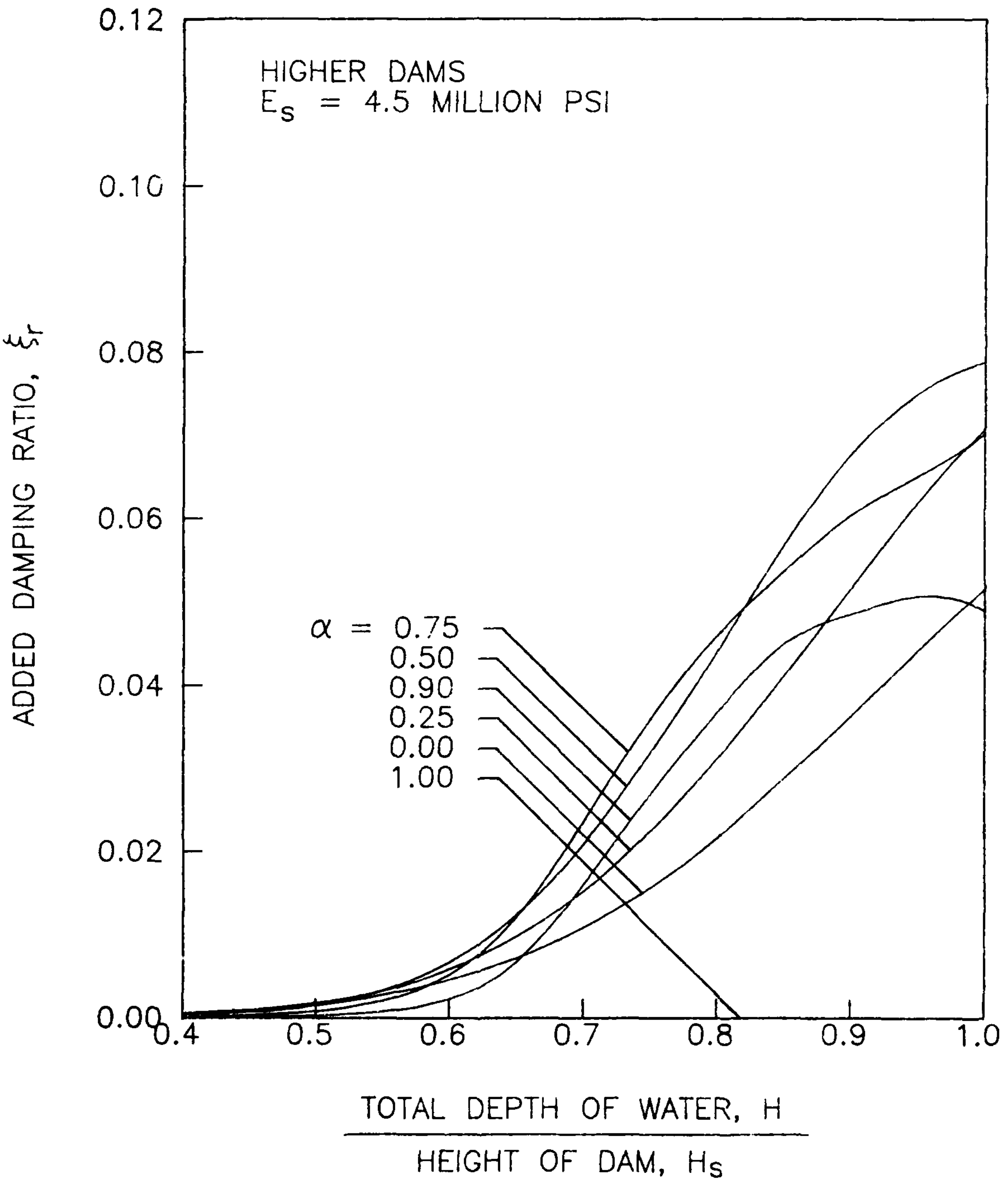

Figure 6(b) -- Standard Values for ξ_r, the Added Damping Ratio due to Hydrodynamic Effects; E_s = 4.5 million psi.

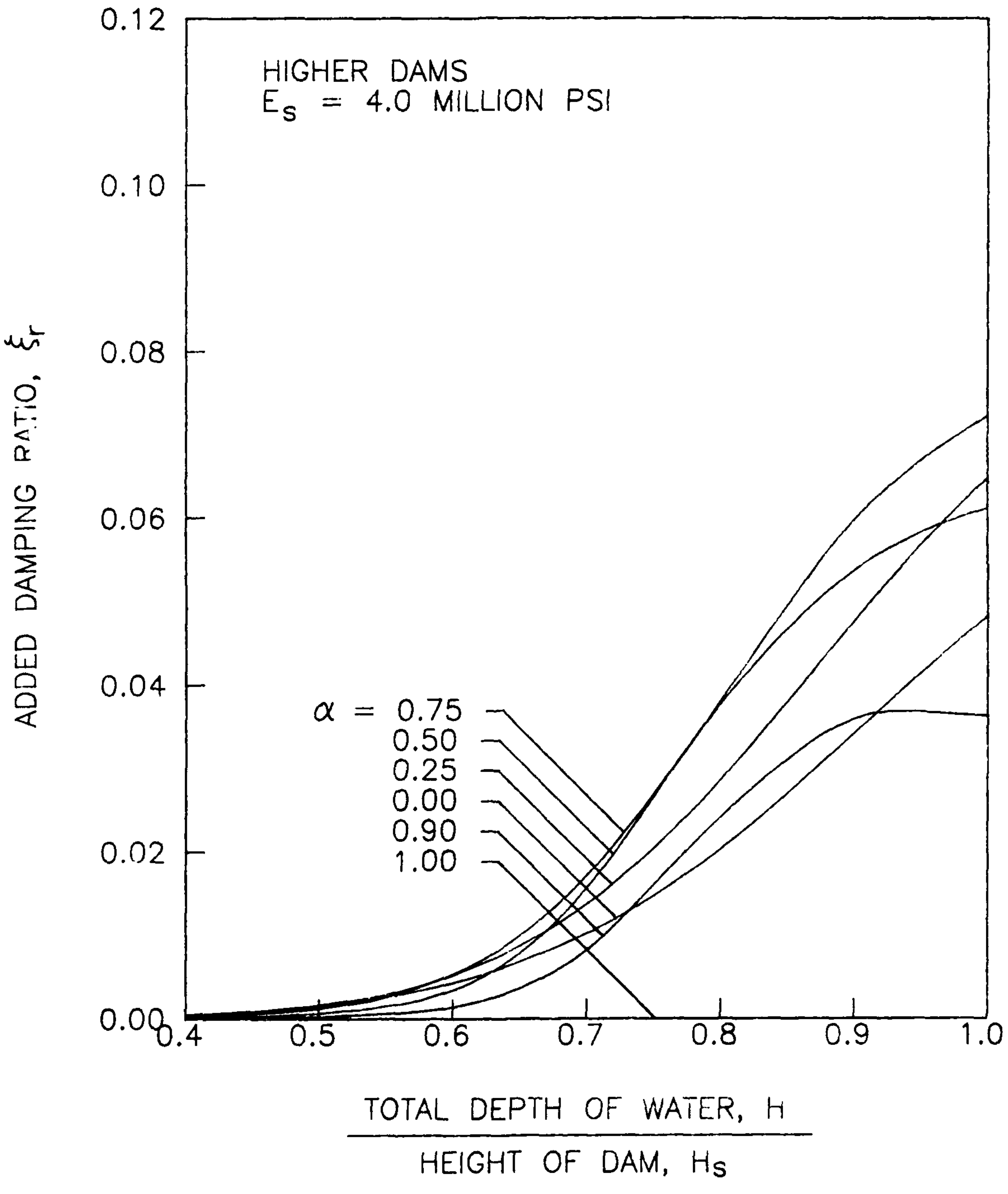

Figure 6(c) -- Standard Values for ξ_r, the Added Damping Ratio due to Hydrodynamic Effects; E_s = 4 million psi.

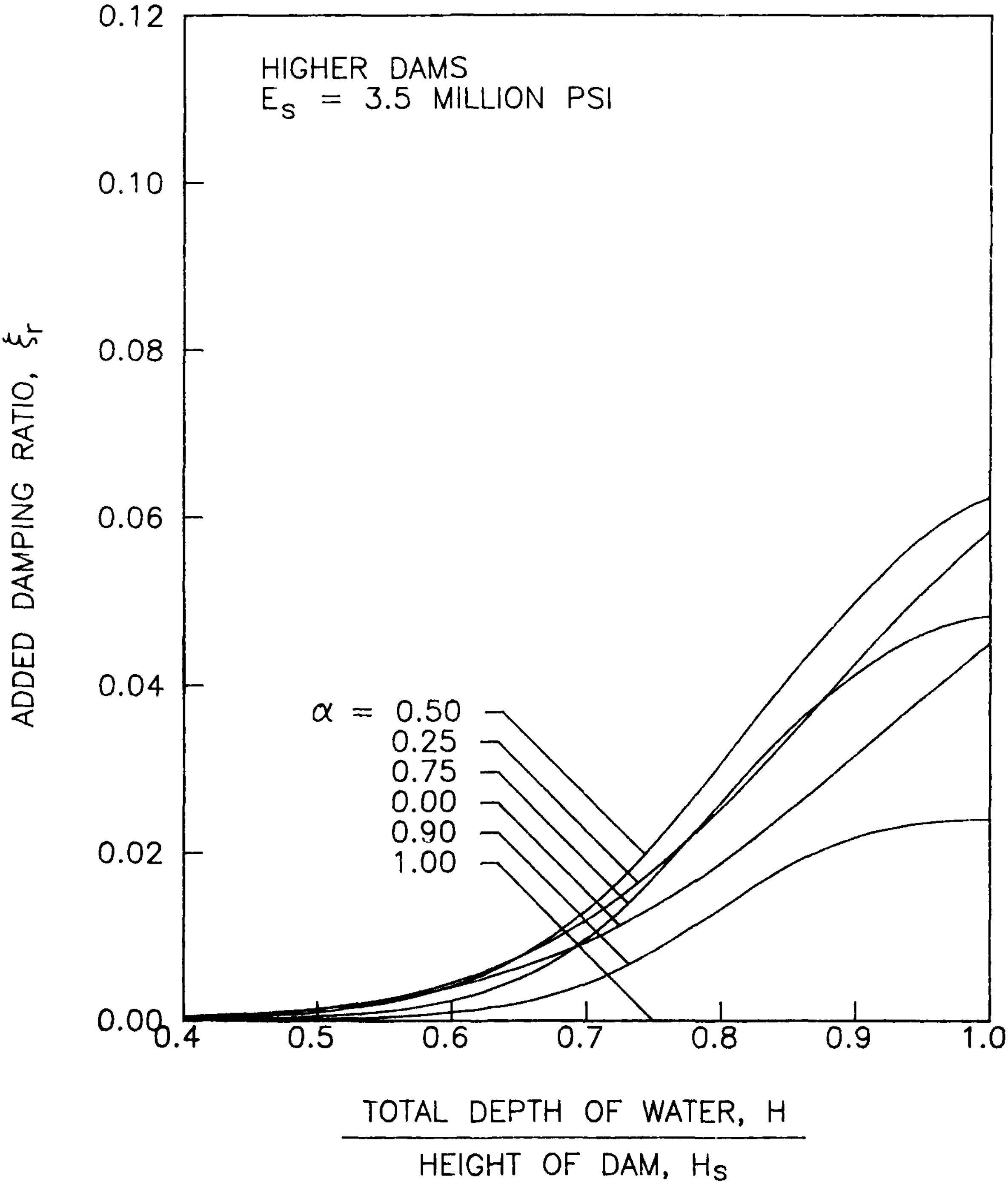

Figure 6(d) -- Standard Values for ξ_r, the Added Damping Ratio due to Hydrodynamic Effects; E_s = 3.5 million psi.

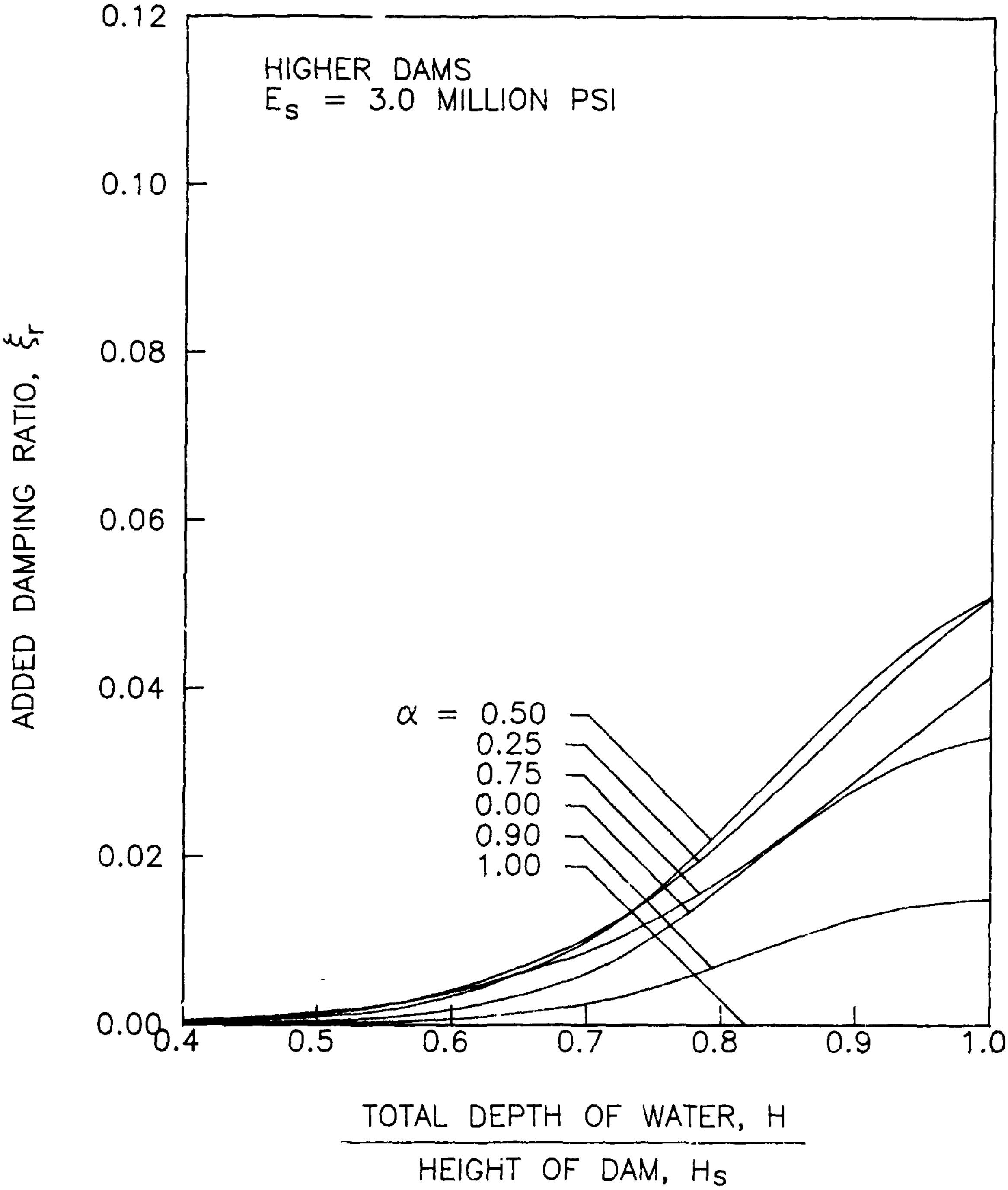

Figure 6(e) -- Standard Values for ξ_r, the Added Damping Ratio due to Hydrodynamic Effects; E_s = 3 million psi.

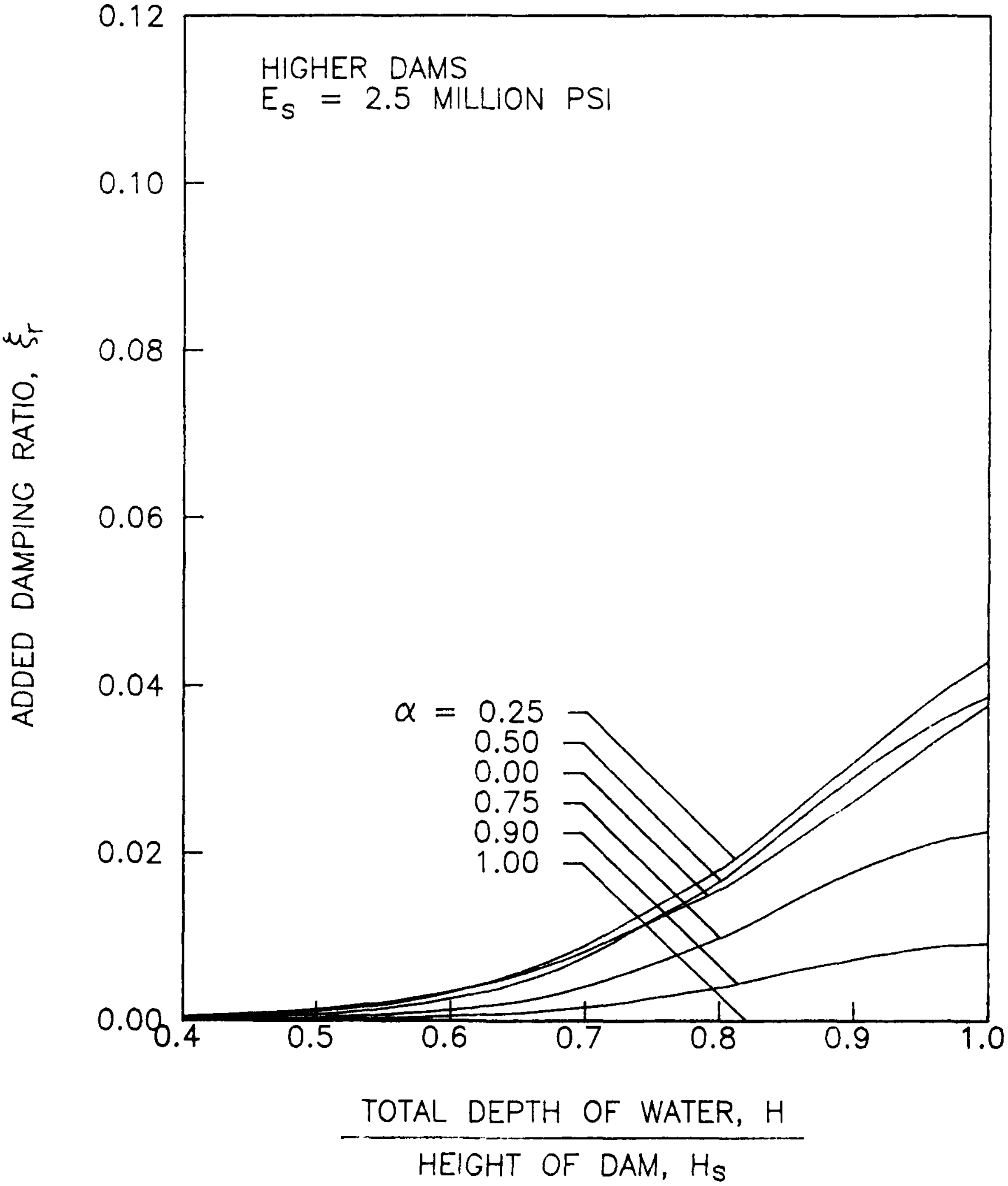

Figure 6(f) -- Standard Values for ξ_r, the Added Damping Ratio due to Hydrodynamic Effects; E_s = 2.5 million psi.

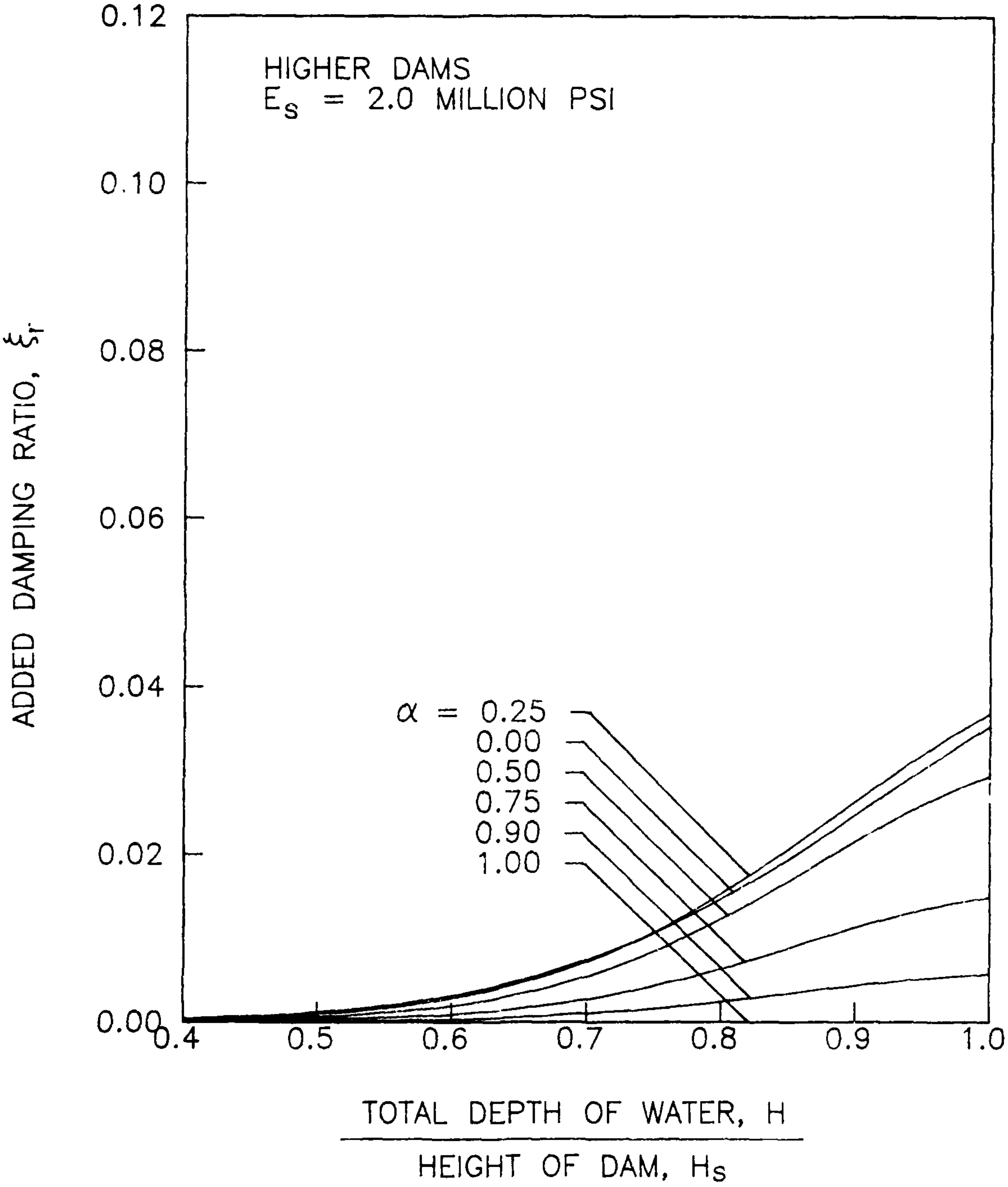

Figure 6(g) -- Standard Values for ξ_r, the Added Damping Ratio due to Hydrodynamic Effects; E_s = 2 million psi.

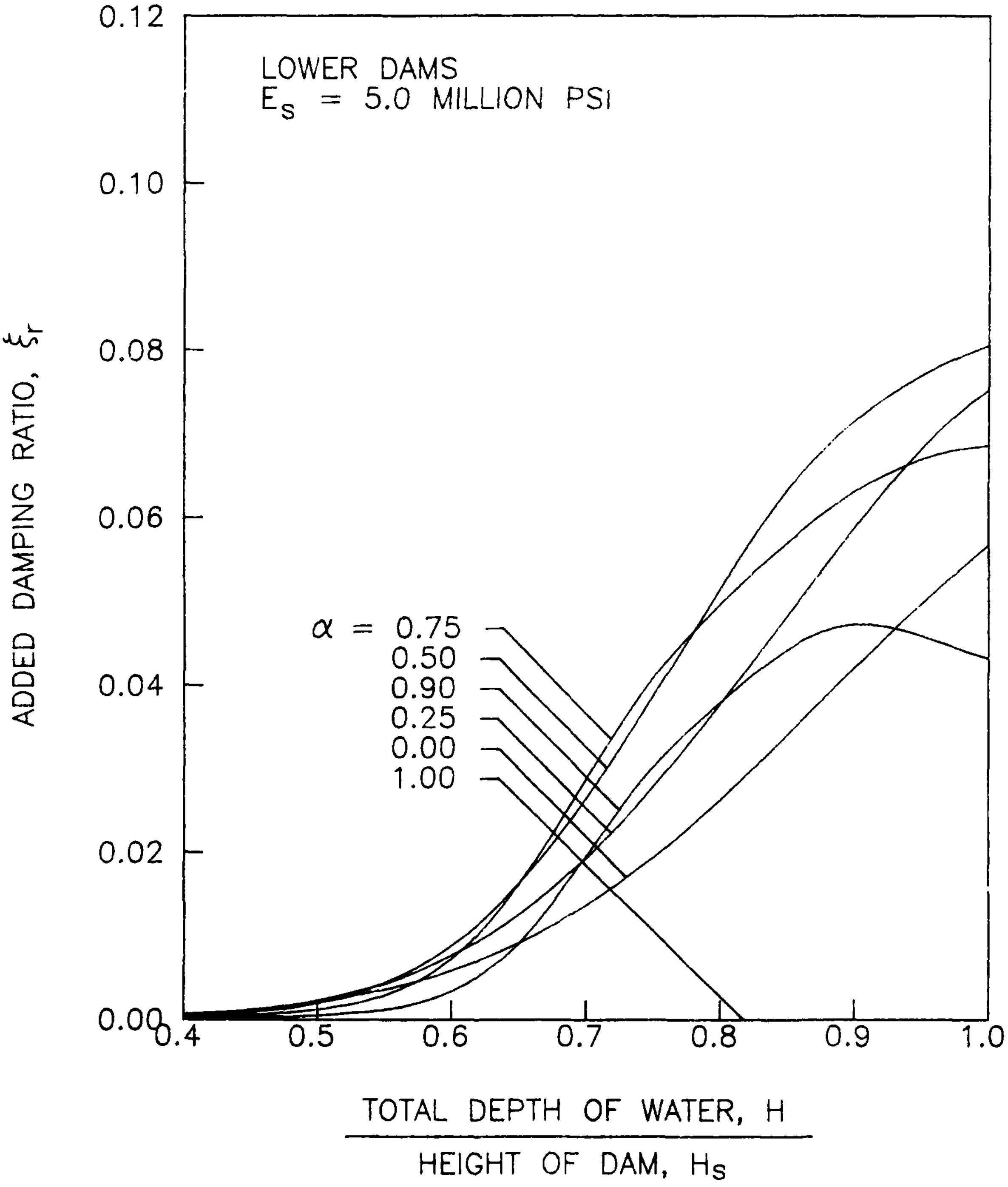

Figure 7(a) –– Standard Values for ξ_r, the Added Damping Ratio due to Hydrodynamic Effects; E_s = 5 million psi.

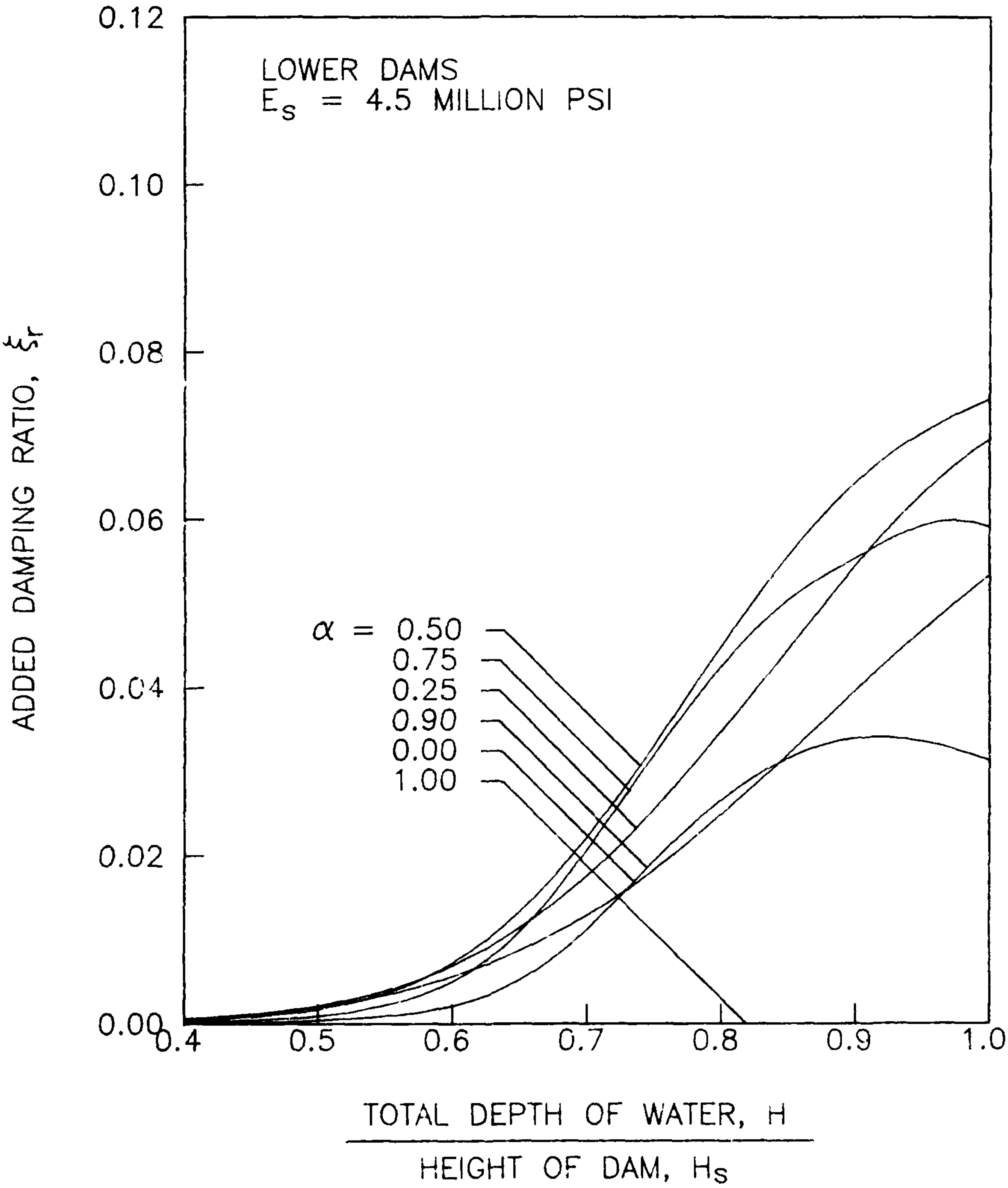

Figure 7(b) -- Standard Values for ξ_r, the Added Damping Ratio due to Hydrodynamic Effects; E_s = 4.5 million psi.

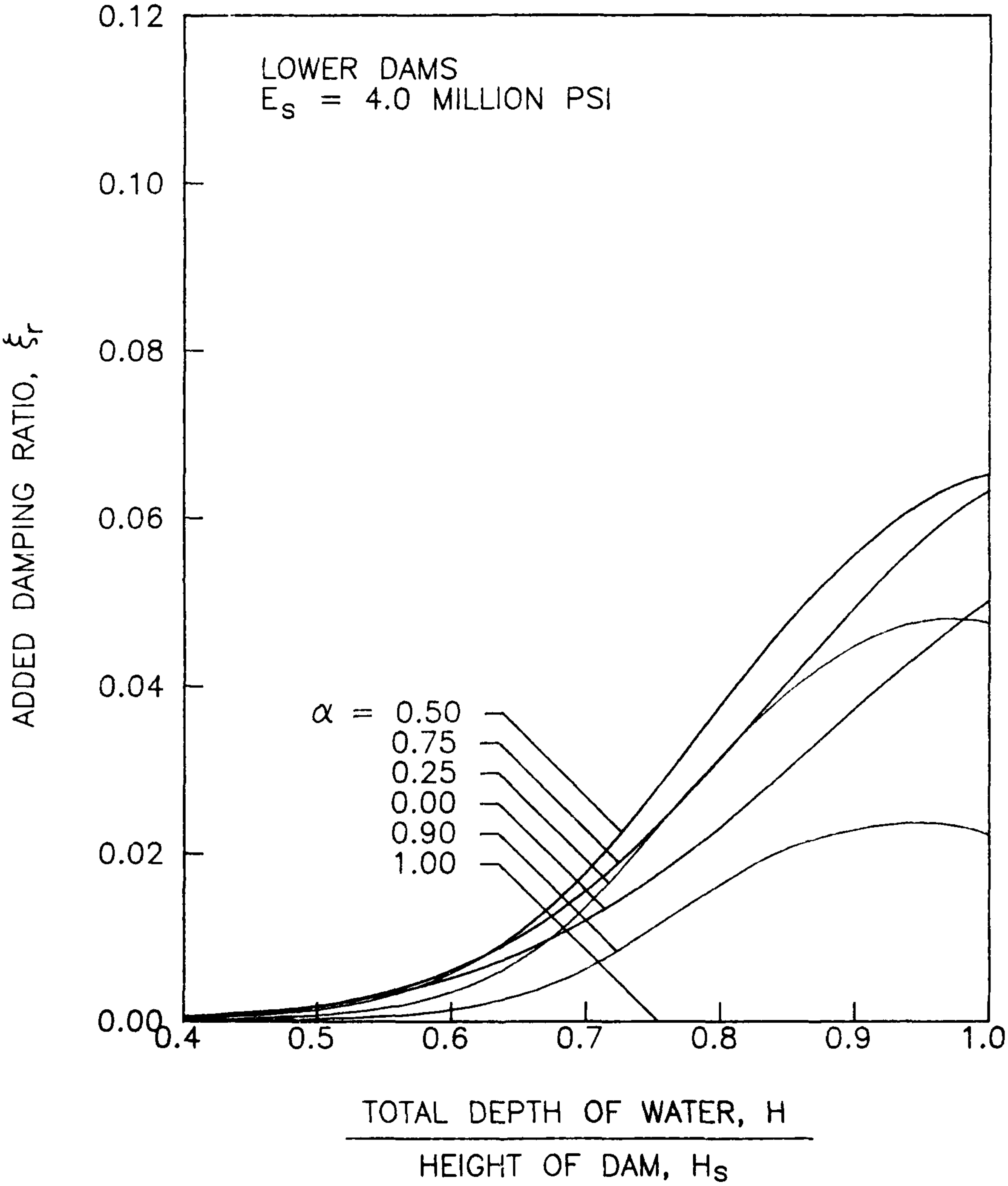

Figure 7(c) -- Standard Values for ξ_r, the Added Damping Ratio due to Hydrodynamic Effects; E_s = 4 million psi.

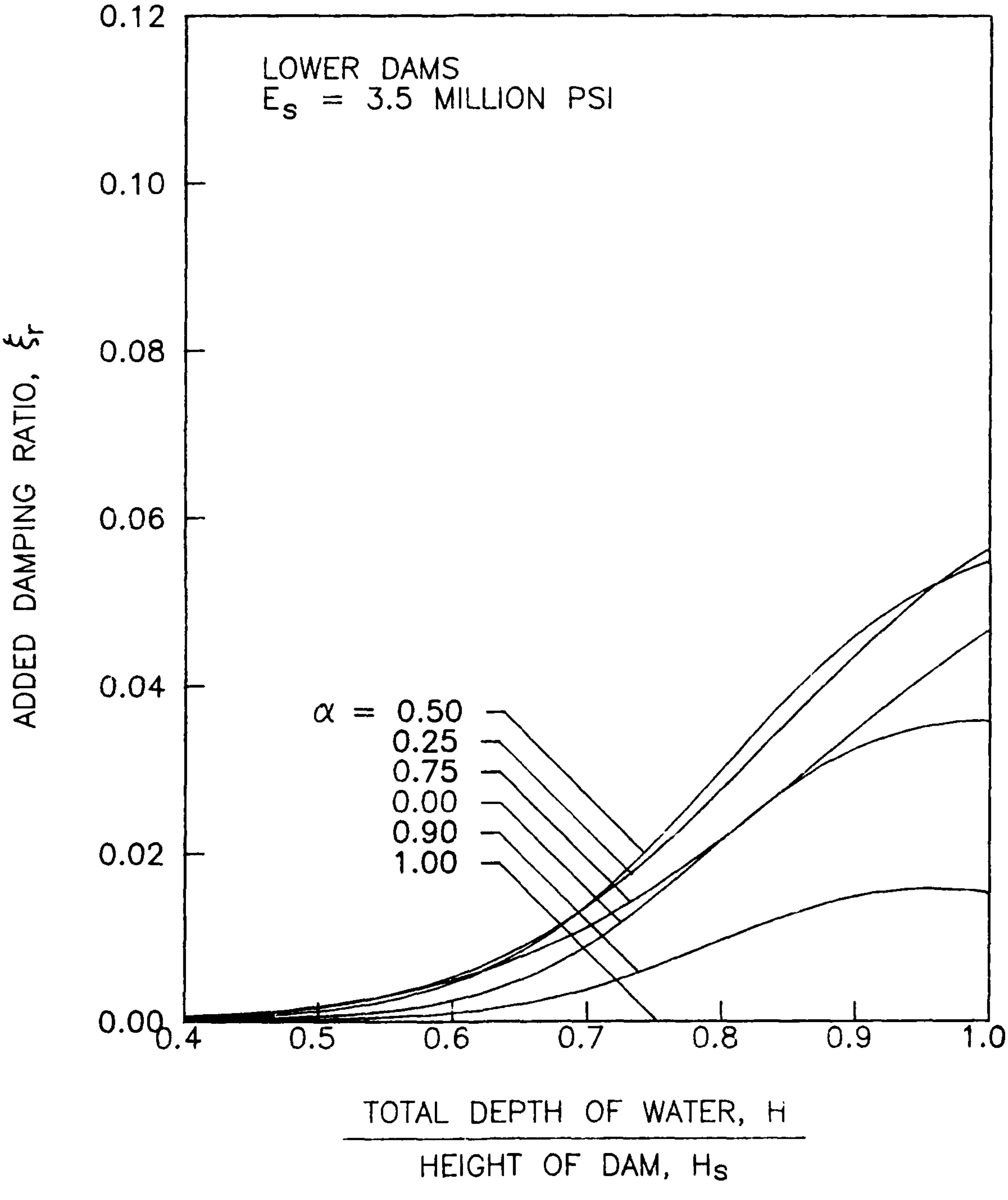

Figure 7(d) -- Standard Values for ξ_r, the Added Damping Ratio due to Hydrodynamic Effects; E_s = 3.5 million psi.

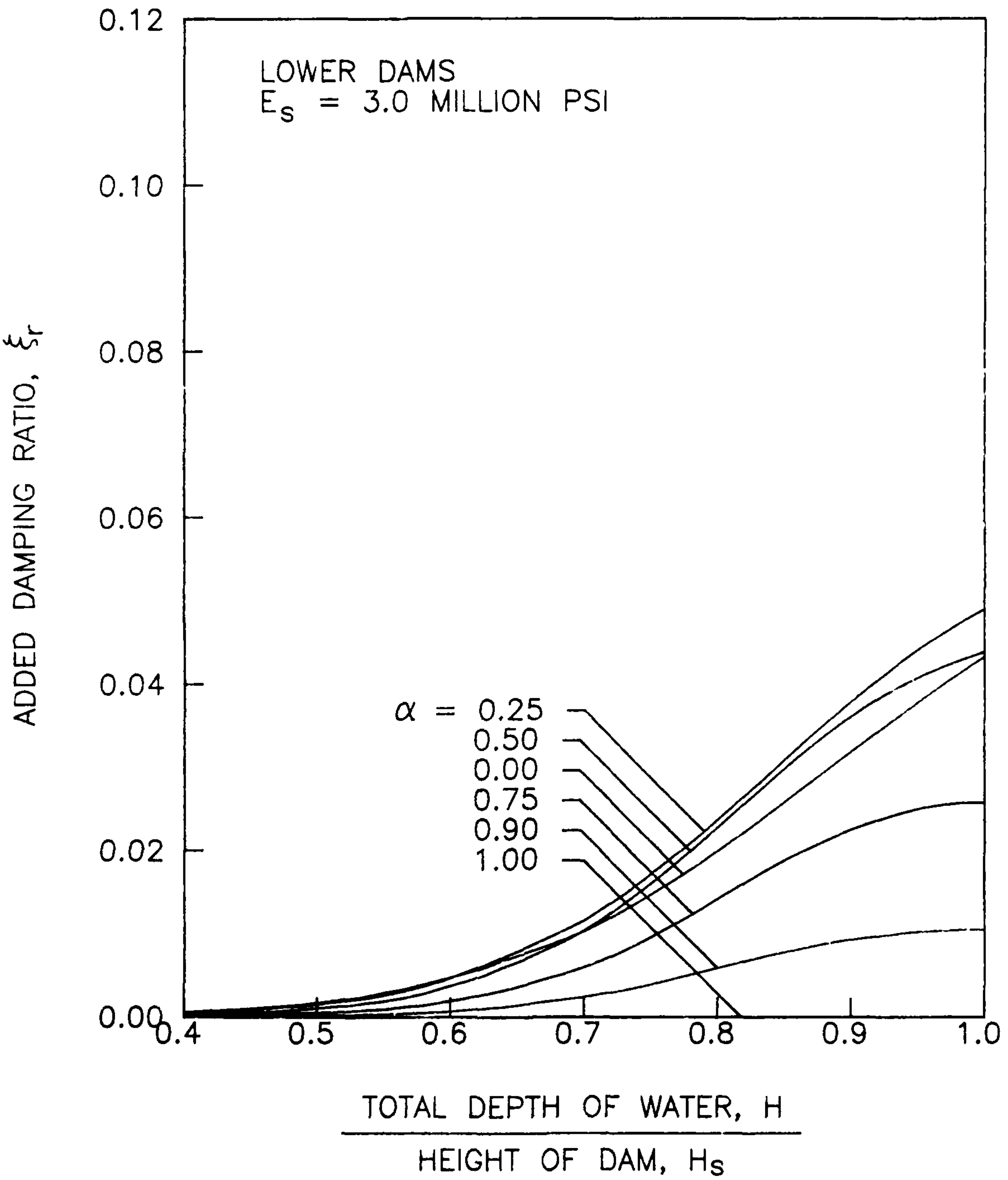

Figure 7(e) -- Standard Values for ξ_r, the Added Damping Ratio due to Hydrodynamic Effects; E_s = 3 million psi.

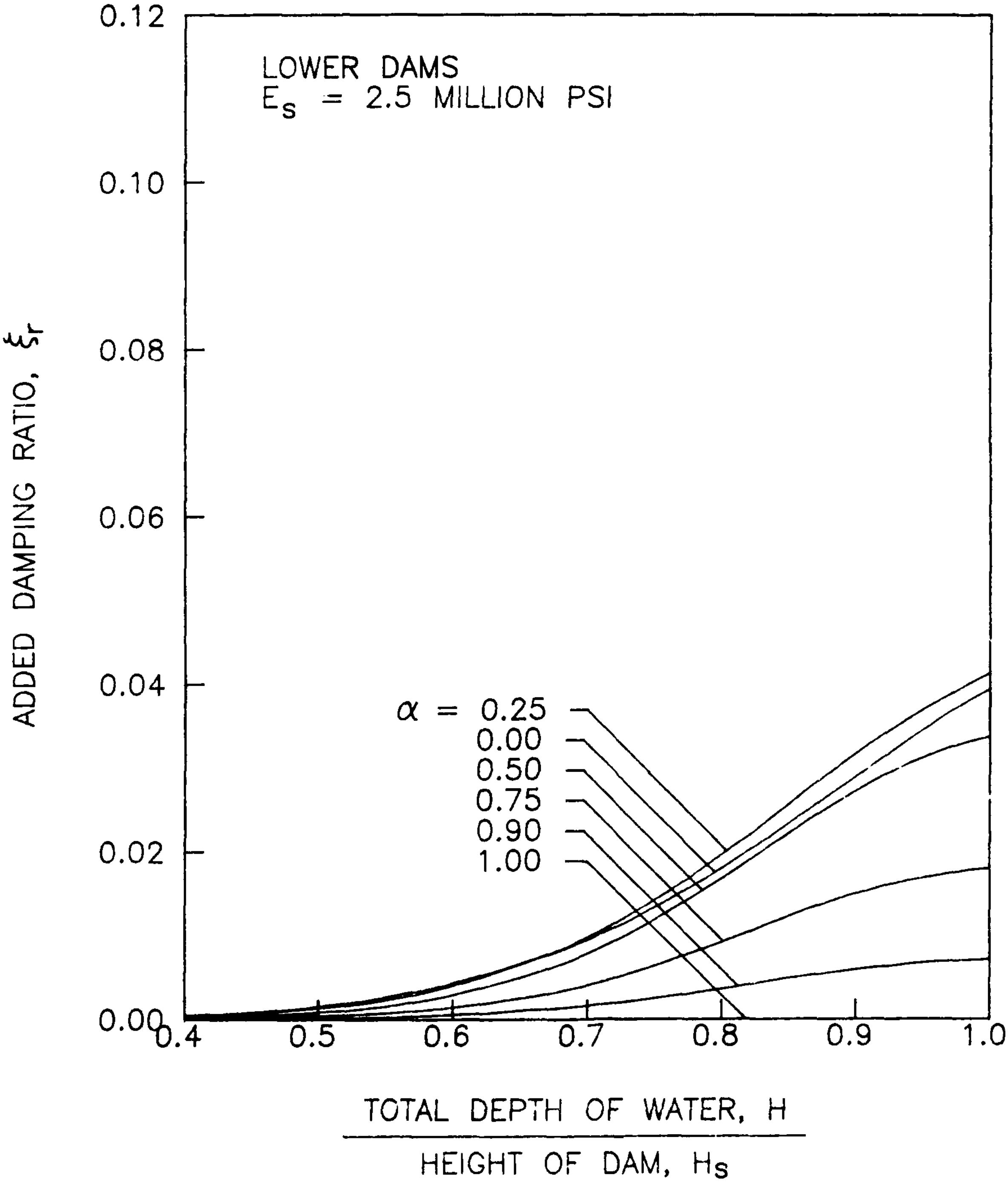

Figure 7(f) -- Standard Values for ξ_r, the Added Damping Ratio due to Hydrodynamic Effects; E_s = 2.5 million psi.

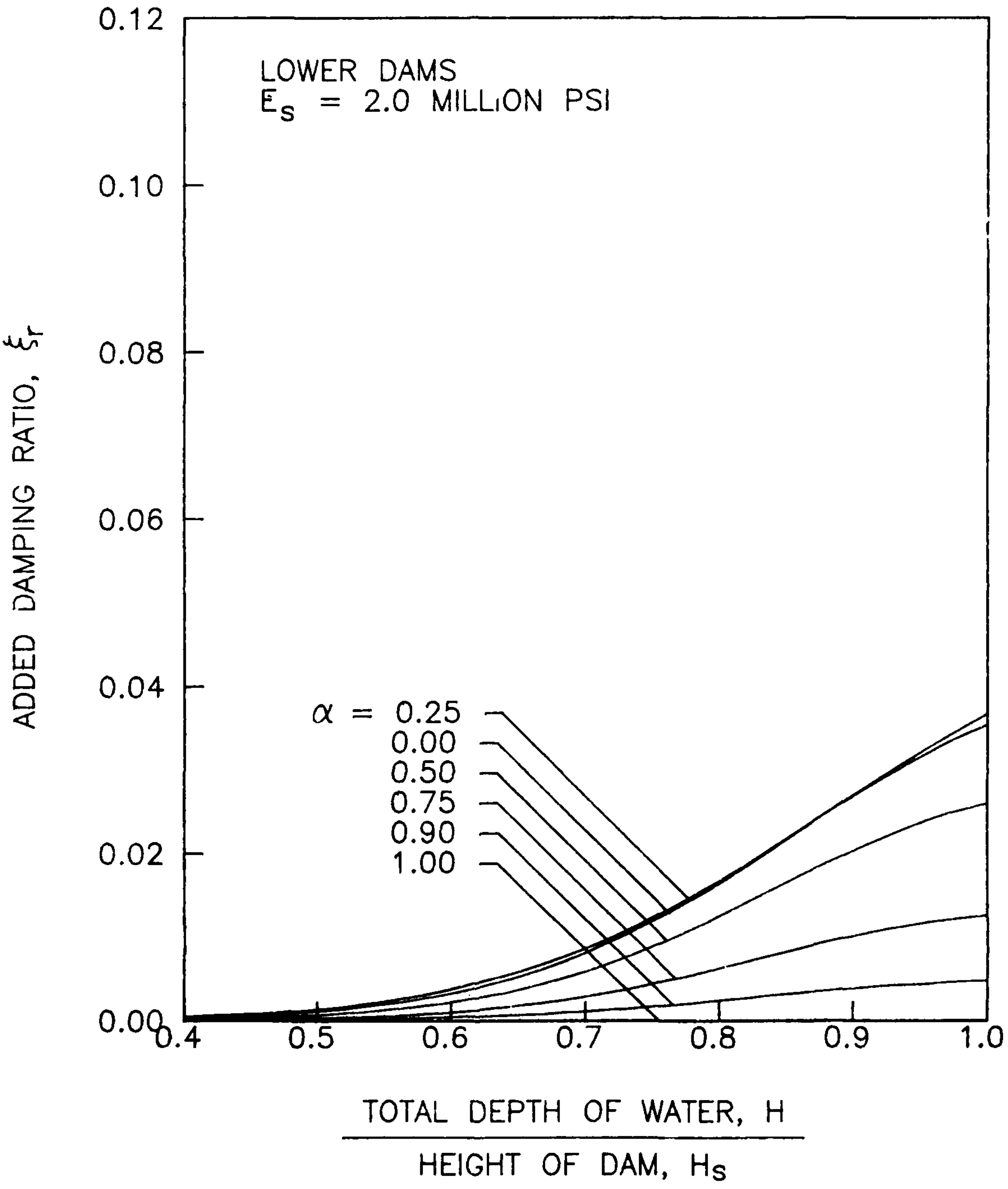

Figure 7(g) -- Standard Values for ξ_r, the Added Damping Ratio due to Hydrodynamic Effects; E_s = 2 million psi.

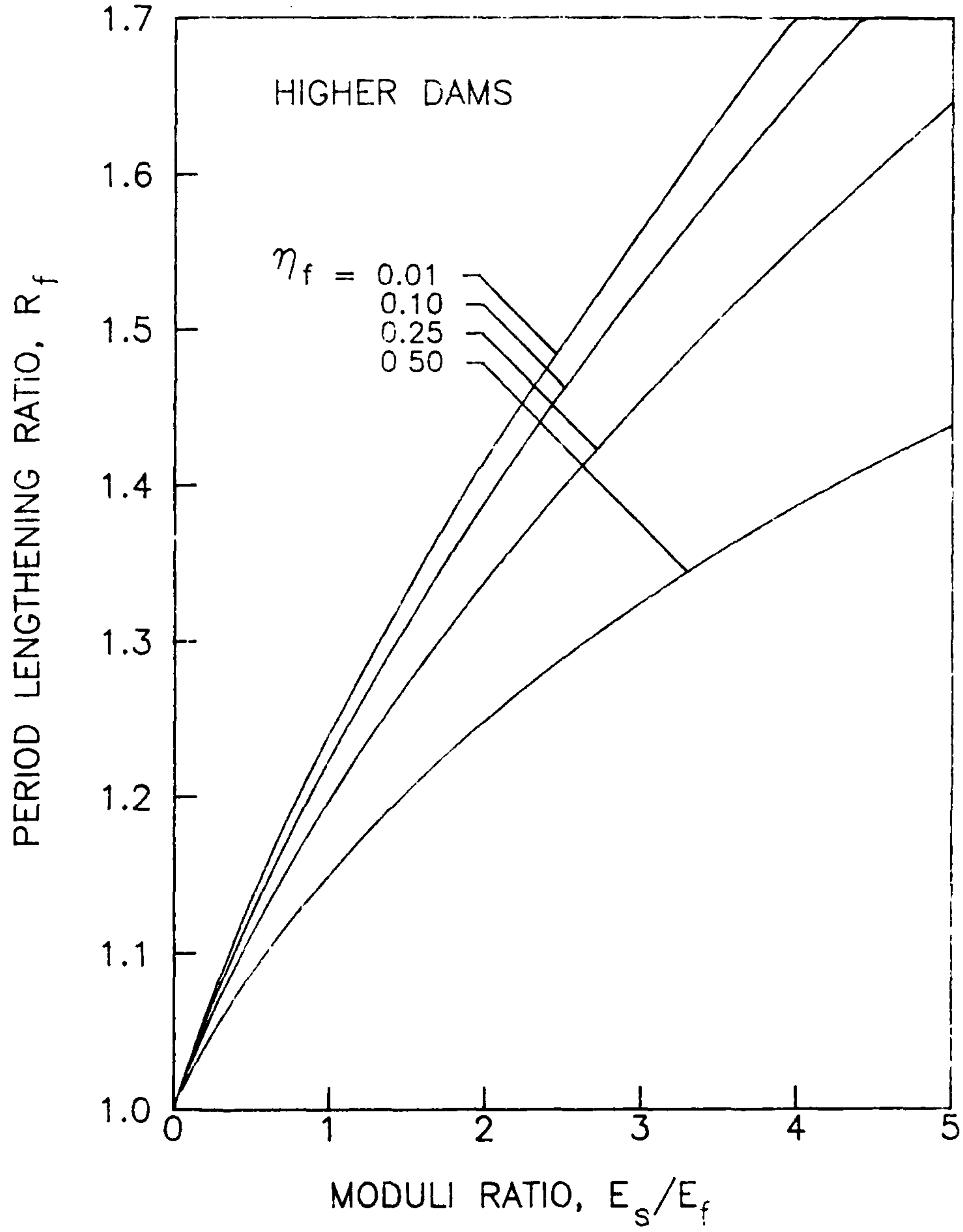

Figure 8 -- Standard Values for R_f, the Period Lengthening Ratio due to Dam–Foundation Rock Interaction

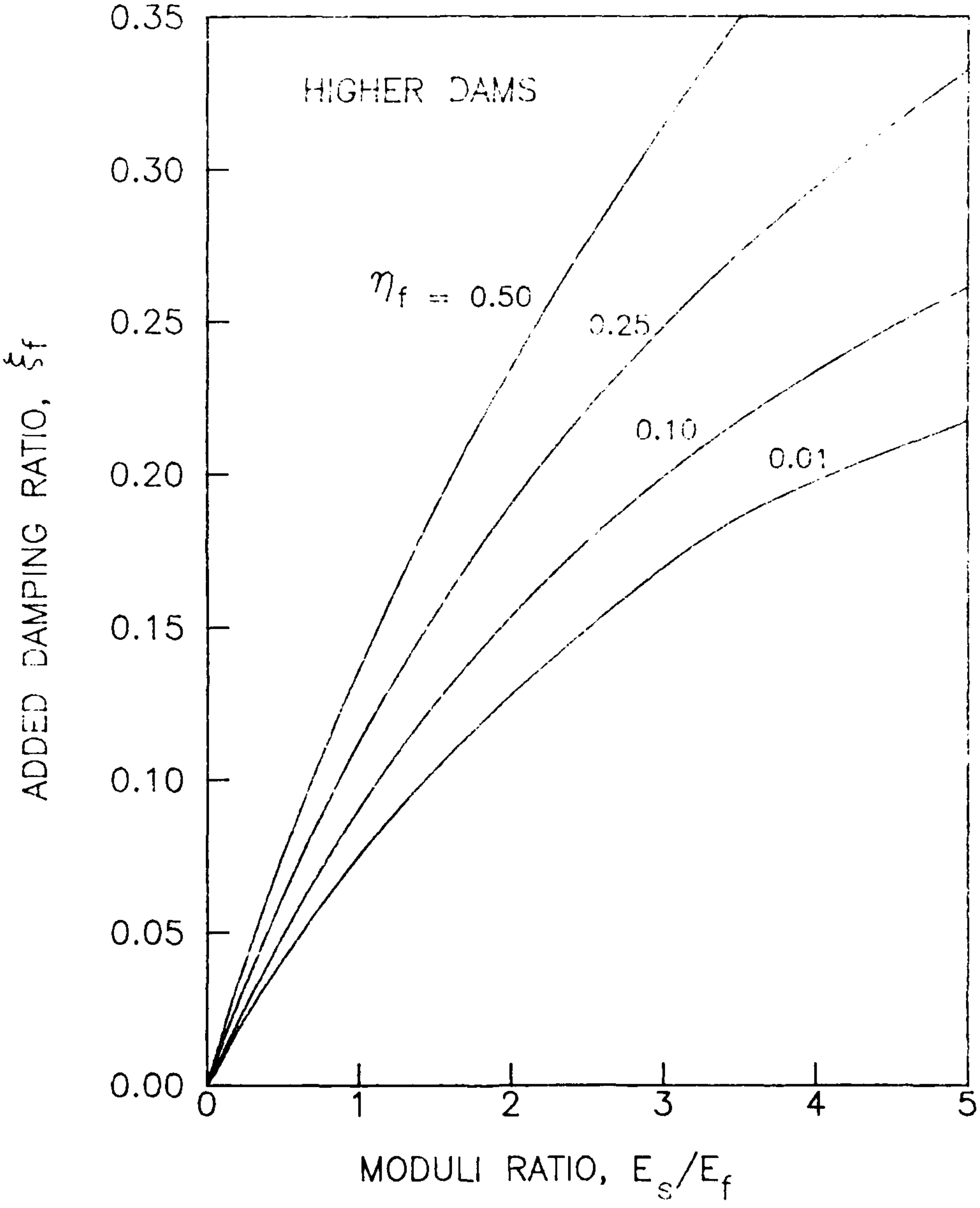

Figure 9 -- Standard Values for ξ_f, the Added Damping Ratio due to Dam-Foundation Rock Interaction

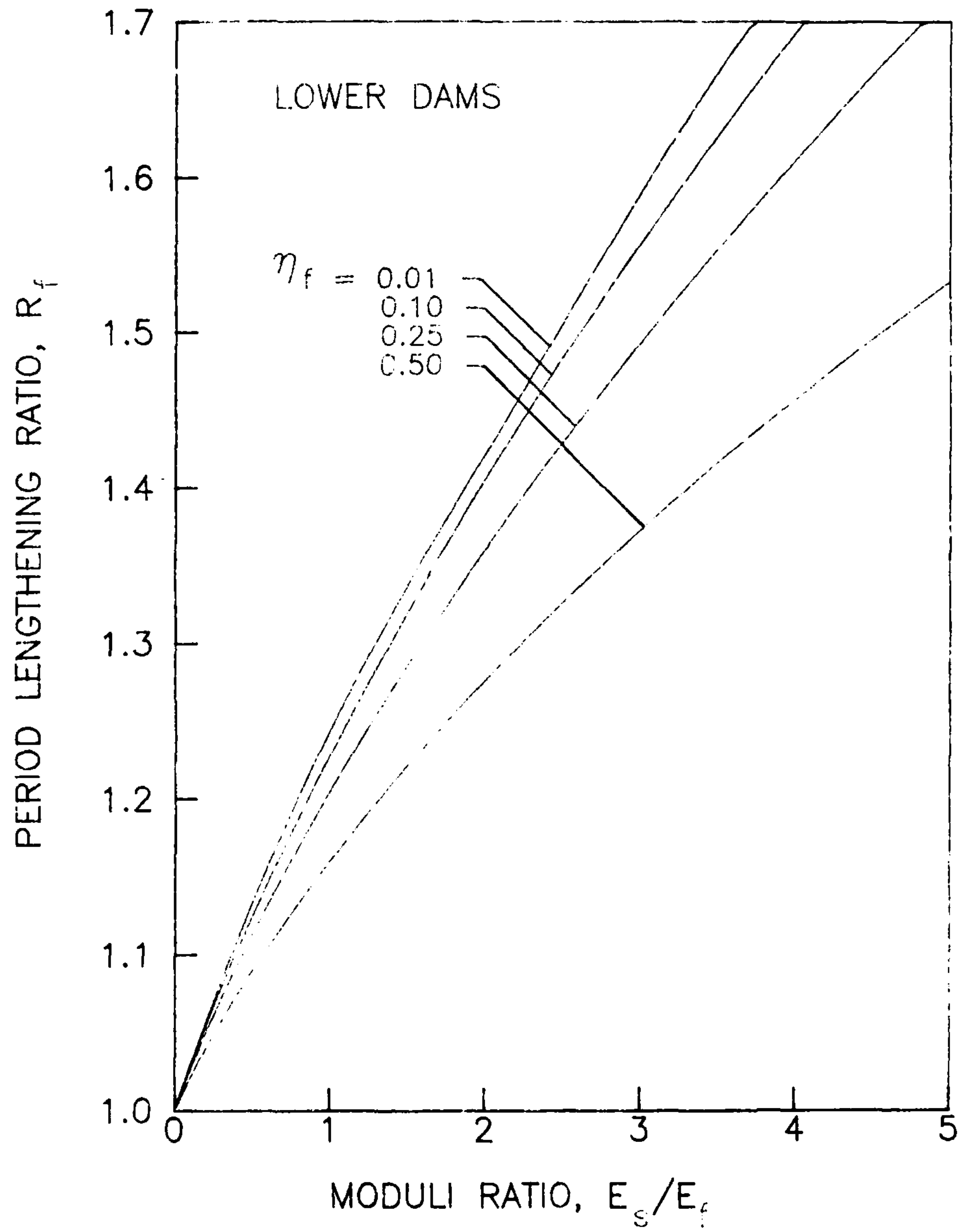

Figure 10 — Standard Values for R_f, the Period Lengthening Ratio due to Dam–Foundation Rock Interaction

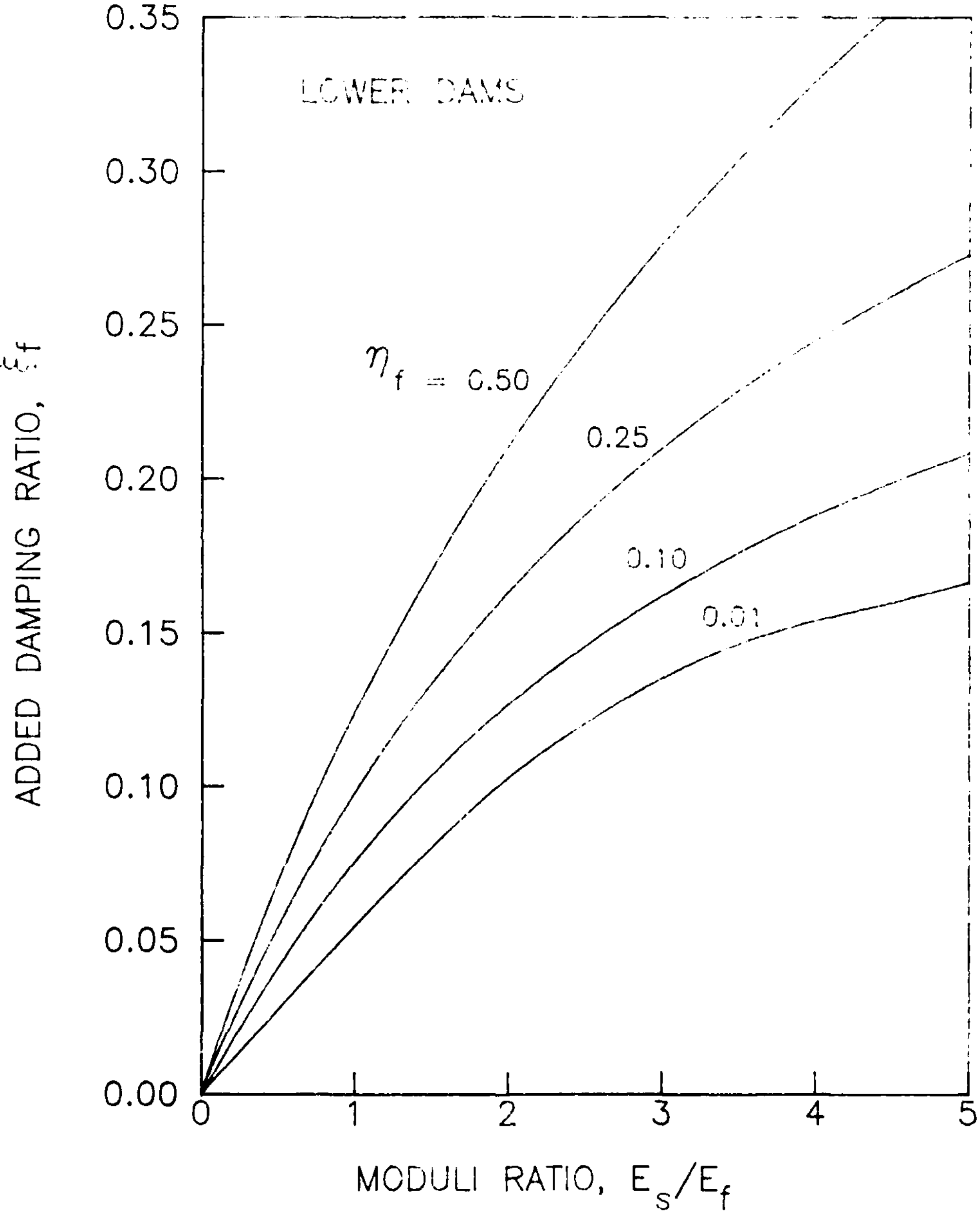

Figure 11 -- Standard Values for ξ_f, the Added Damping Ratio due to Dam-Foundation Rock Interaction

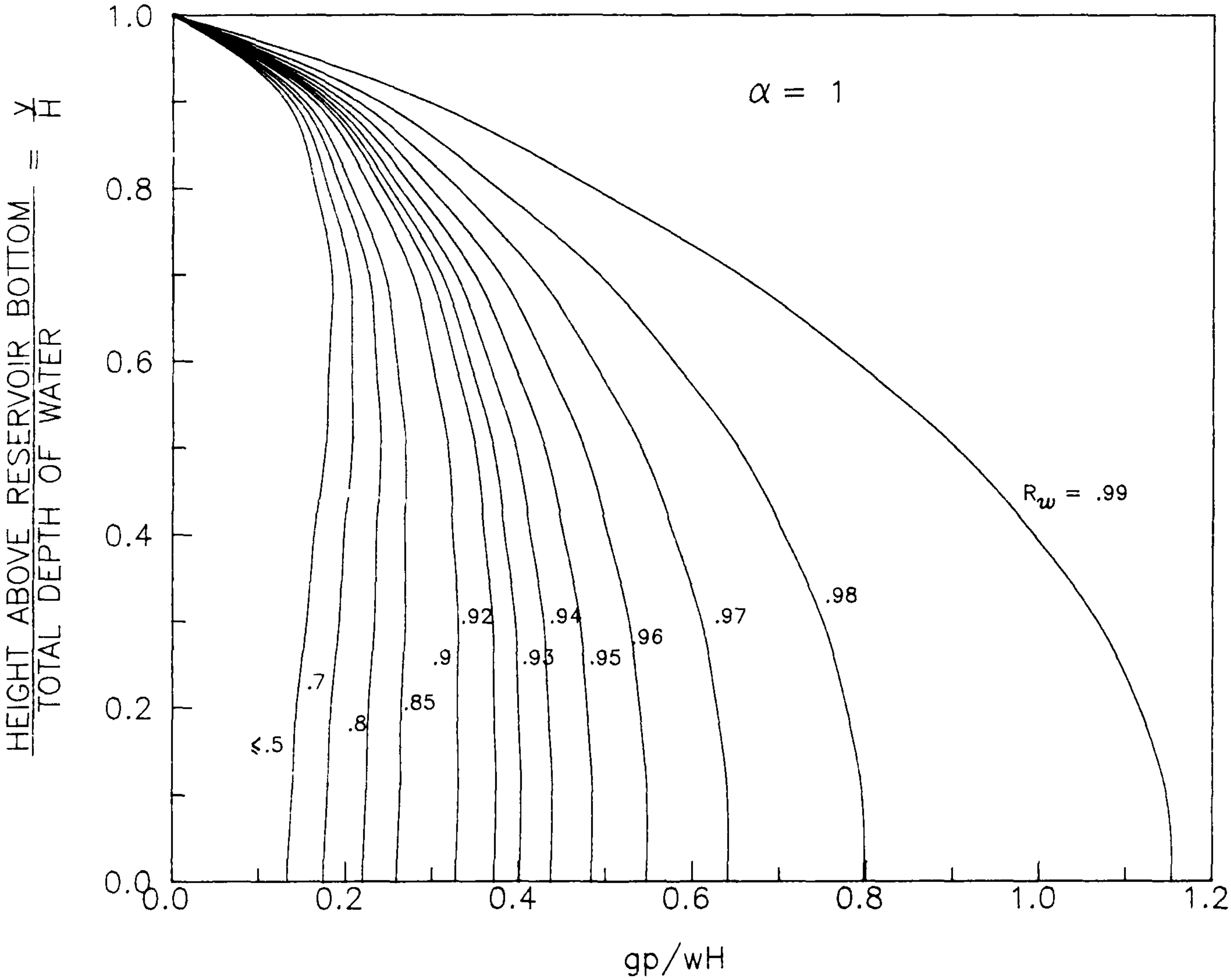

Figure 12(a) -- Standard Values for the Hydrodynamic Pressure Function p(y/H) for Full Reservoir, i.e., $H/H_s = 1$; $\alpha = 1.00$ -- Higher and Lower Dams

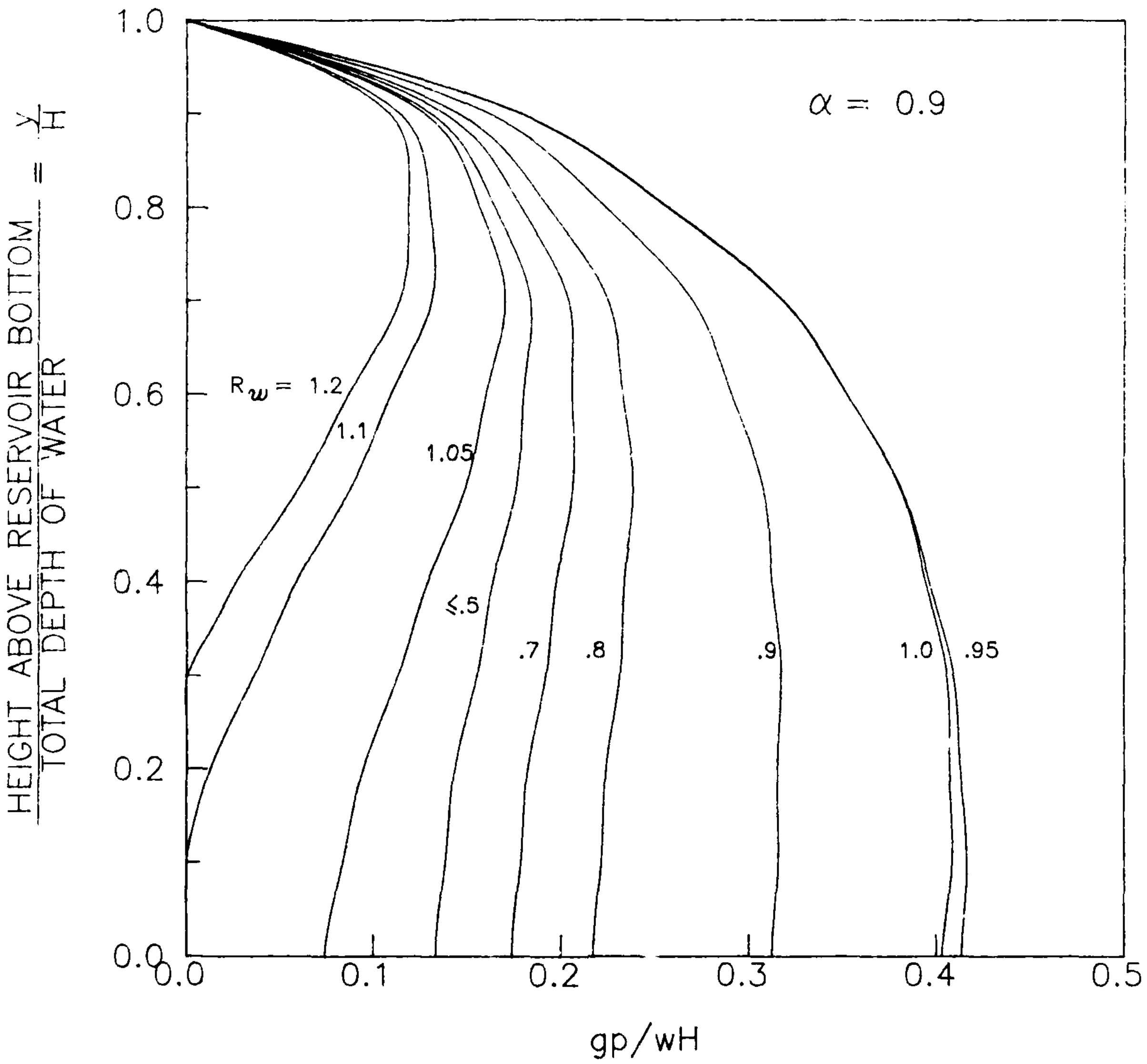

Figure 12(b) -- Standard Values for the Hydrodynamic Pressure Function $p(y/H)$ for Full Reservoir, i.e., $H/H_s = 1$; $\alpha = 0.90$ -- Higher and Lower Dams

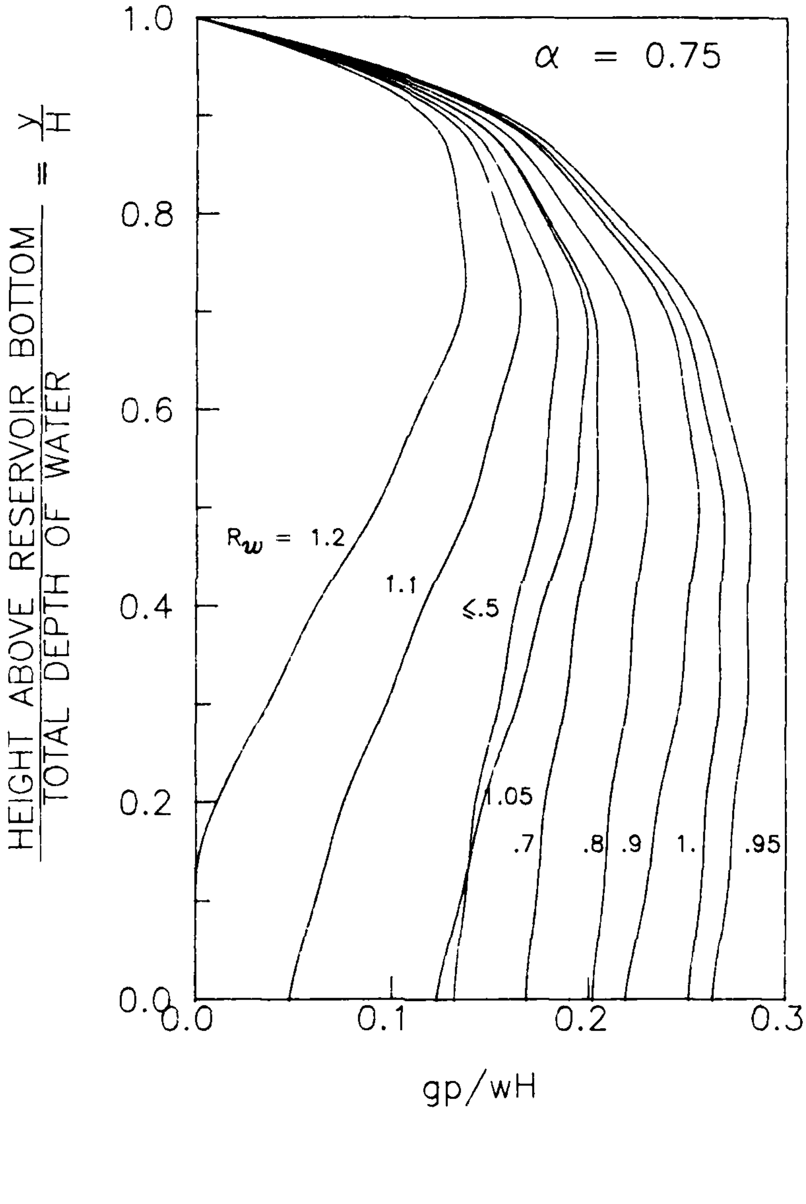

Figure 12(c) -- Standard Values for the Hydrodynamic Pressure Function p(y/H) for Full Reservoir, i.e., $H/H_s = 1$; $\alpha = 0.75$ -- Higher and Lower Dams

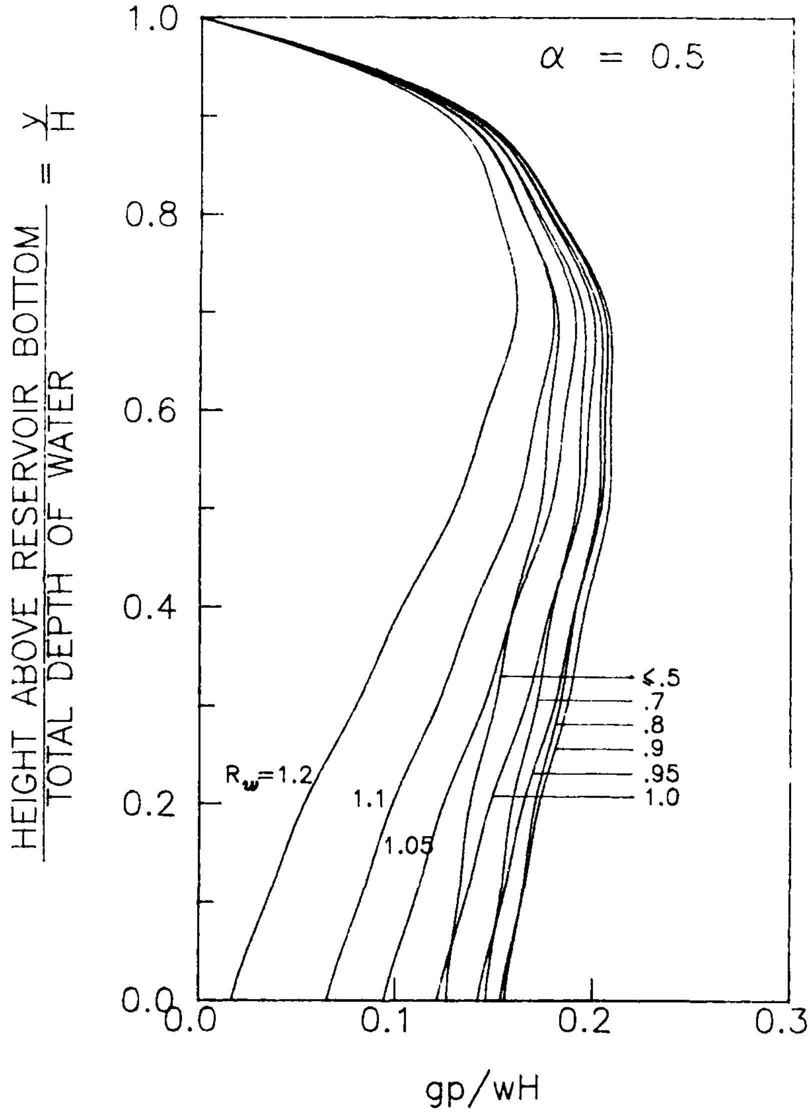

Figure 12(d) -- Standard Values for the Hydrodynamic Pressure Function p(y/H) for Full Reservoir, i.e., H/H_s = 1; α = 0.50 -- Higher and Lower Dams

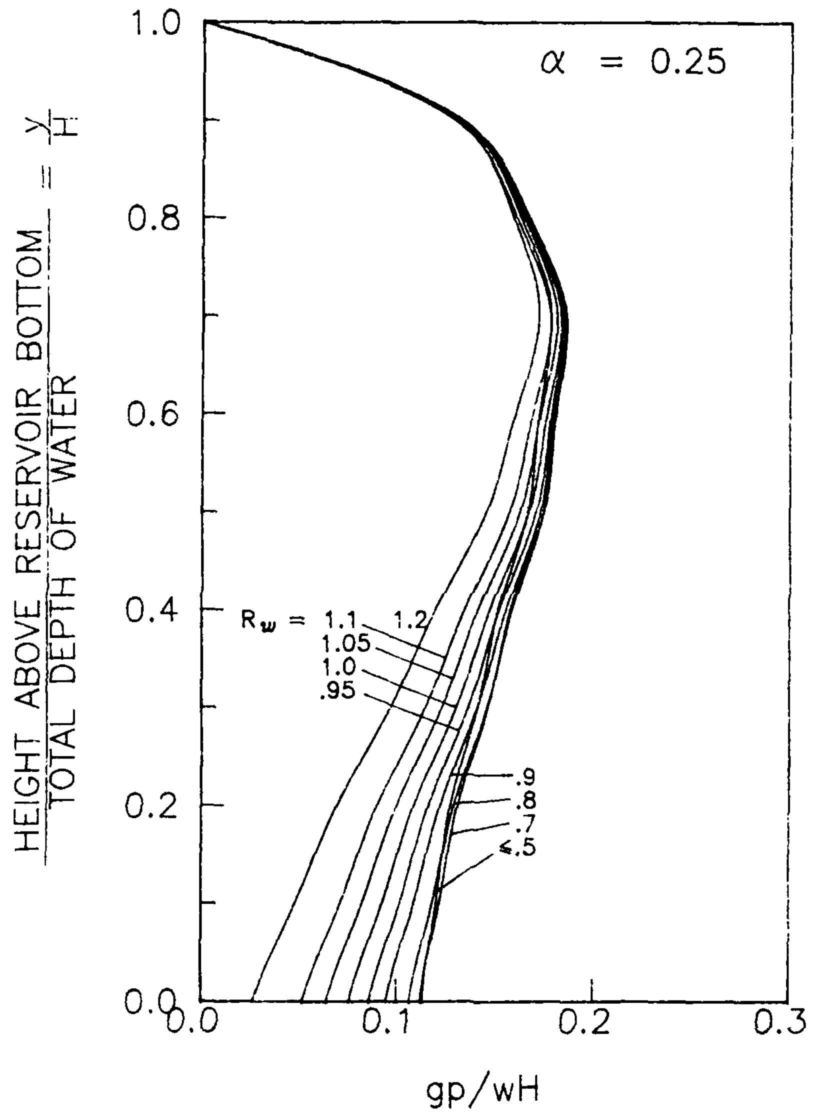

Figure 12(e) -- Standard Values for the Hydrodynamic Pressure Function $p(y/H)$ for Full Reservoir, i.e., $H/H_S = 1$; $\alpha = 0.25$ -- Higher and Lower Dams

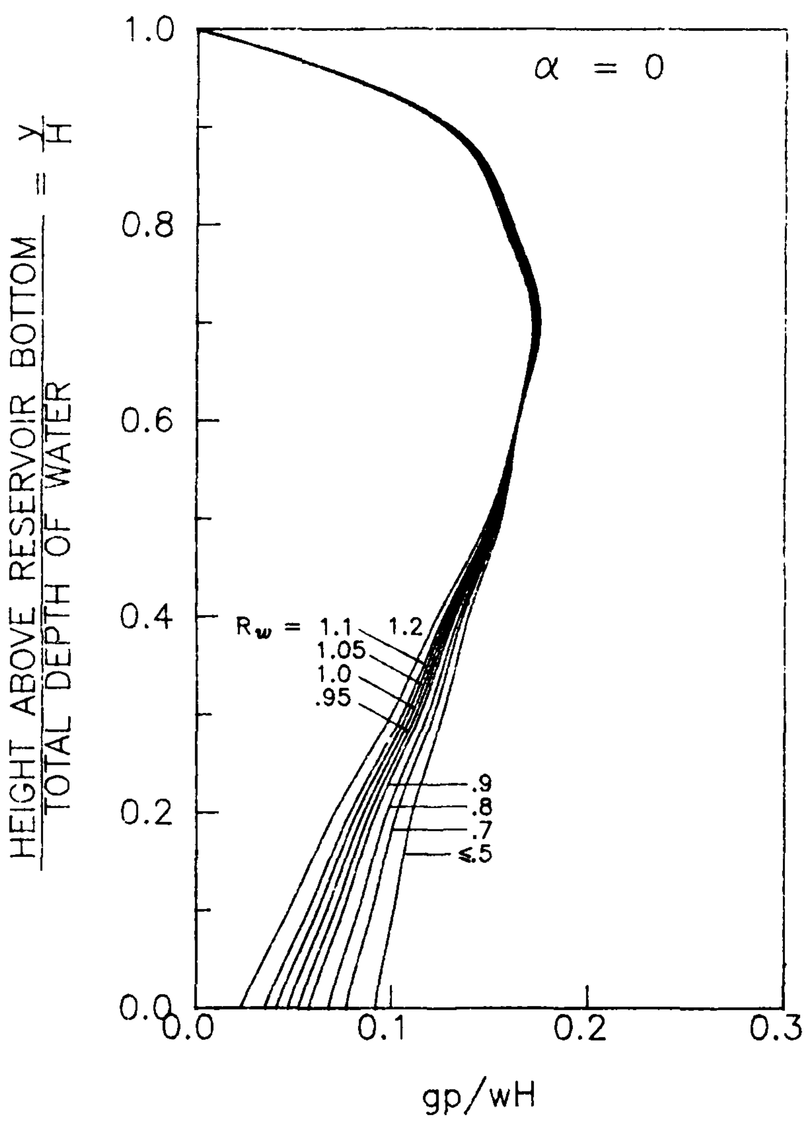

Figure 12(f) -- Standard Values for the Hydrodynamic Pressure Function $p(y/H)$ for Full Reservoir, i.e., $H/H_s = 1$; $\alpha = 0.00$ -- Higher and Lower Dams

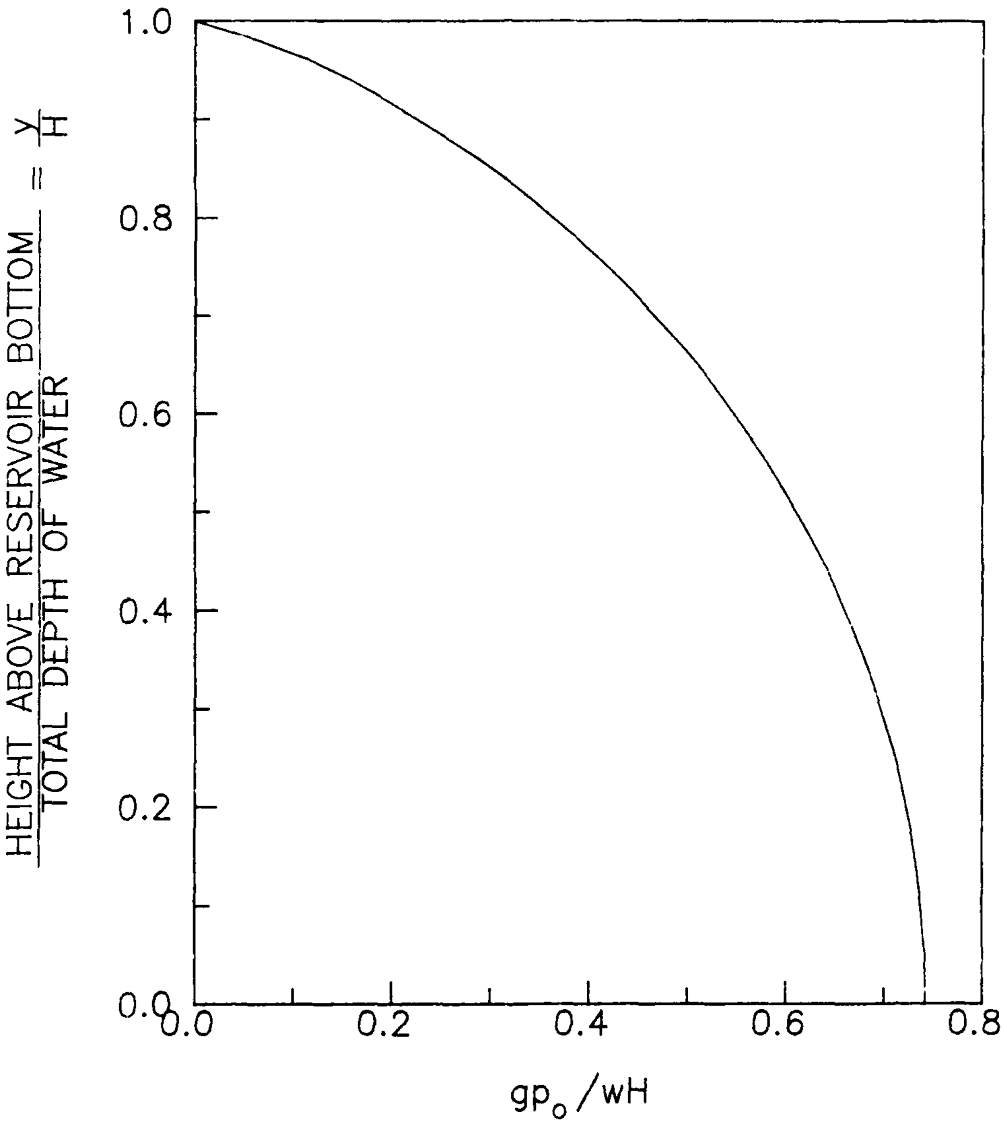

Figure 13 -- Standard Values for the Hydrodynamic Pressure Function $p_o(y)$

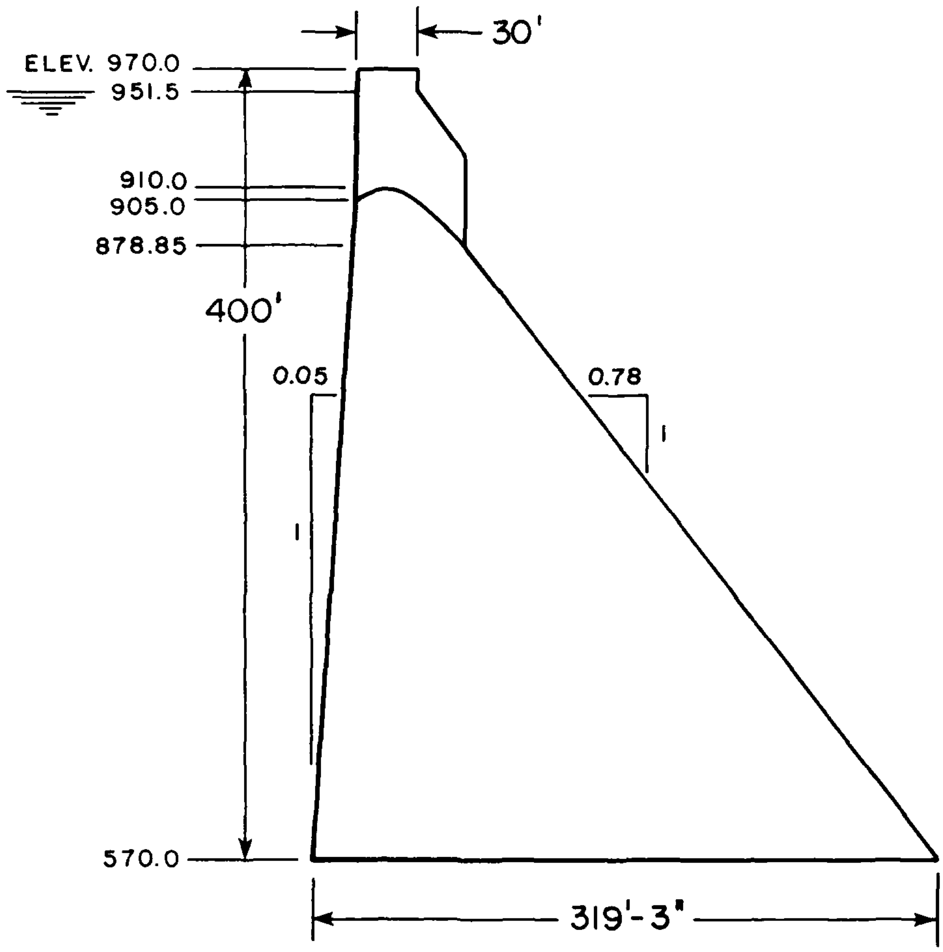

Figure 14 -- Pine Flat Dam: Tallest Spillway Monolith and Pier

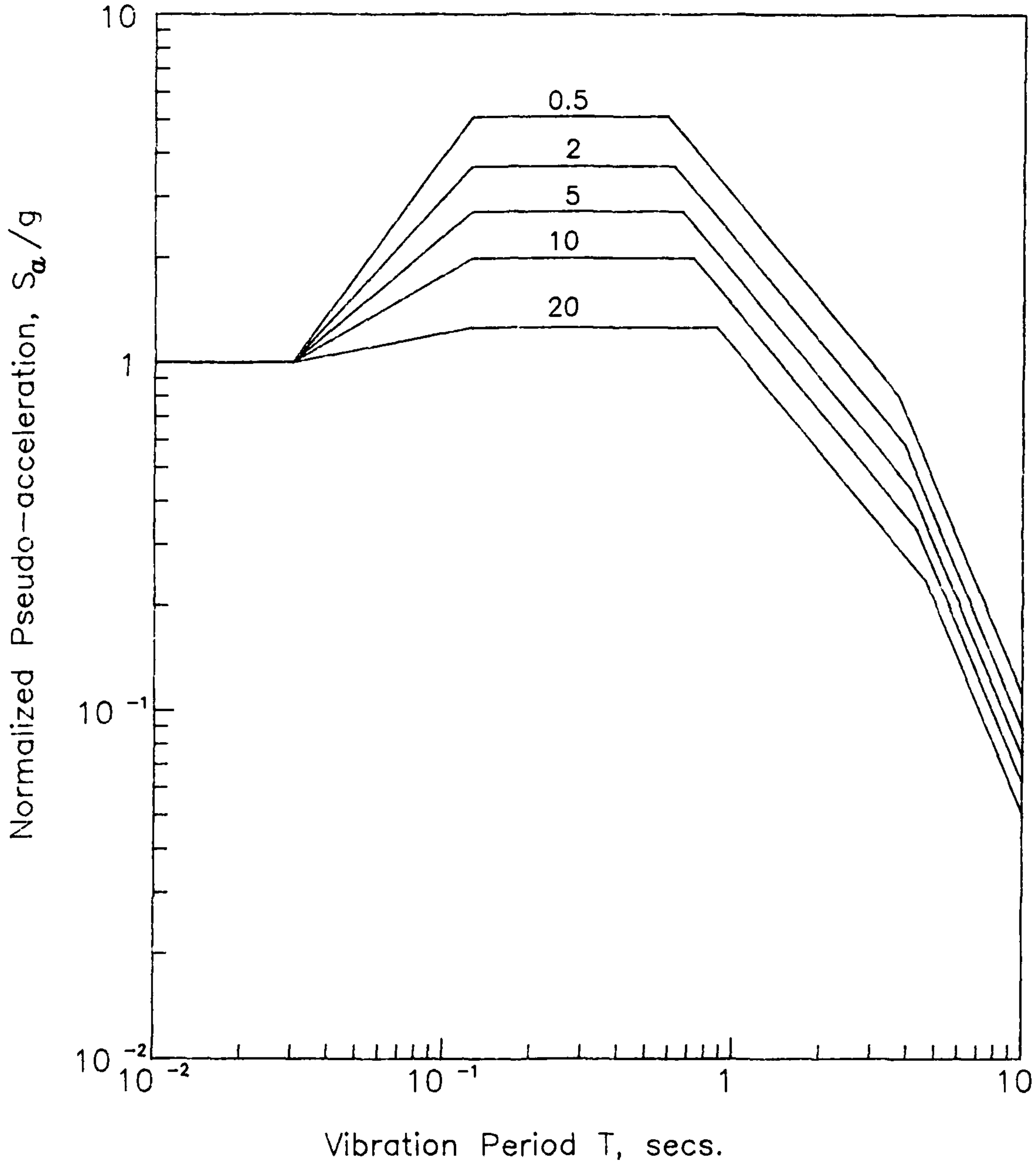

Figure 15 -- Elastic Design Spectrum, Horizontal Motion,
One Sigma Cumulative Probability,
Damping Ratios = 0.5, 2, 5, 10, and 20 percent

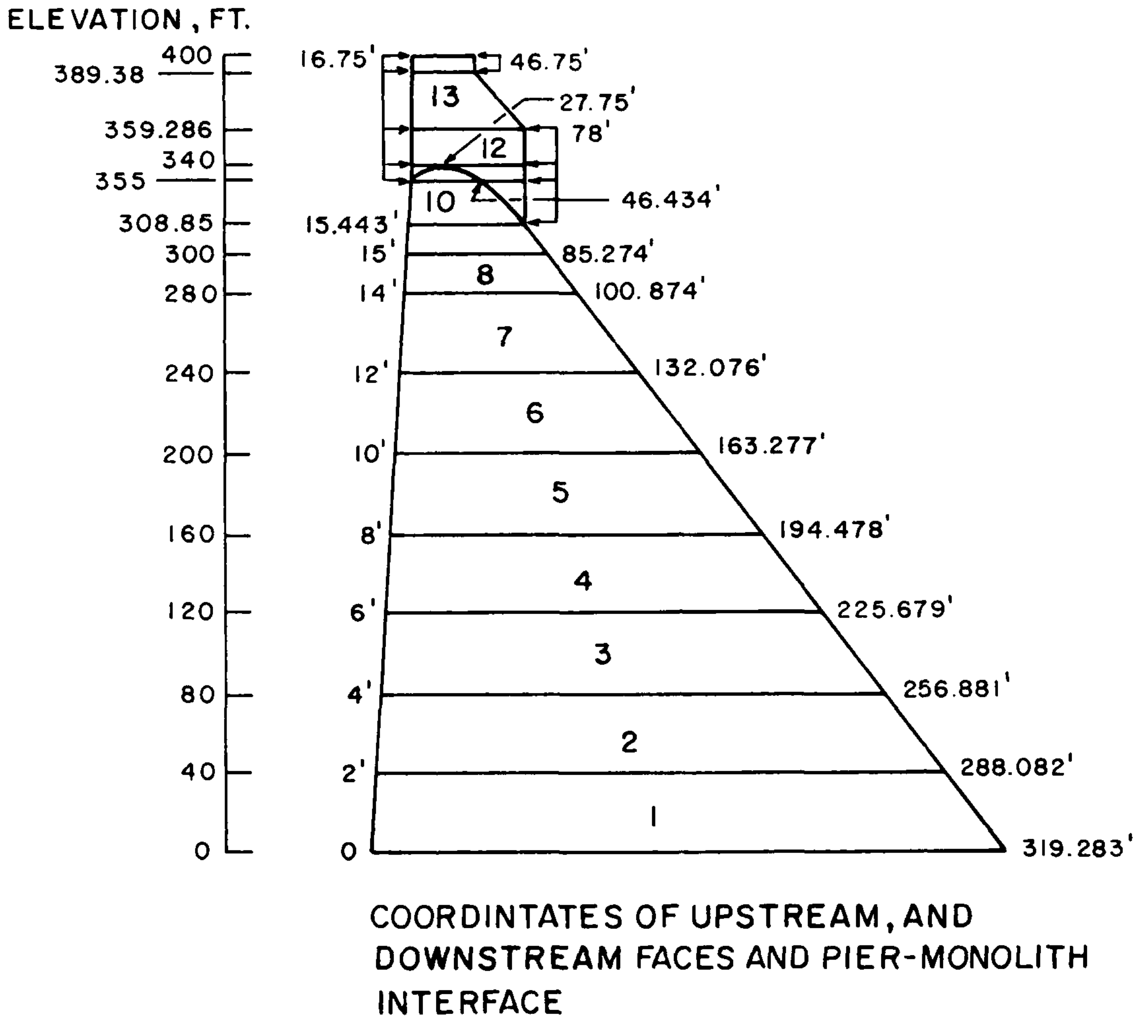

Figure 16 -- Block Model of Spillway of Pine Flat Dam

APPENDIX A: STANDARD SPILLWAY CROSS-SECTIONS

The spillway monoliths of Pine Flat Dam and Richard B. Russel Dam are idealized as shown in Fig. A.1. Computations carried out at the writer's request by Dr. R. L. Hall, Waterways Experiment Station, Corps of Engineers, have demonstrated that the bridge, gate, and foot bucket may be neglected in estimating the fundamental vibration period and mode shape. Therefore, these components have not been included in the structural idealization. Also, the pier geometry has been somewhat simplified in the idealization. The height H_s is 400 ft for the tallest monolith of Pine Flat Dam (Fig. A.1(a)) and 200 ft in the case of Russell Dam (Fig. A.1(b)). However, the cross-section of Fig. A.1(a) is representative of higher dams, i.e. $H_s = 300$ to 600 ft, and Fig. A.1(b) is typical of lower dams, i.e. $H_s = 0$ to 300 ft.

The fundamental vibration period and mode shape of dams (monolith plus pier) of Fig. A.1 were computed for various values of H_s, with the size and shape of the pier unchanged. Because the widths of the monolith and pier along the dam axis are 50 ft and 10 ft, respectively, it may seem that a three-dimensional analysis is required. However, computations carried out by Dr. R. L. Hall at the writer's request led to the following conclusion: An equivalent two-dimensional system of unit thickness along the dam axis, with the unit weight and elastic modulus of the pier reduced by a factor equal to the ratio of the monolith width to pier width, is satisfactory for computing the fundamental vibration period and mode shape of a dam. Finite element idealizations of these two-dimensional systems were analyzed by the EAGD-84 computer program. In these analyses the Poisson's ratio of the dam concrete was taken as 0.2.

The resulting fundamental vibration periods and mode shapes are presented in Fig. A.2 for dams of Fig. A.1(a) for four heights H_s between 300 and 600 ft. Expressing the computed fundamental vibration period as

$$T_1 = \beta \frac{H_s}{\sqrt{E}} \tag{A.1}$$

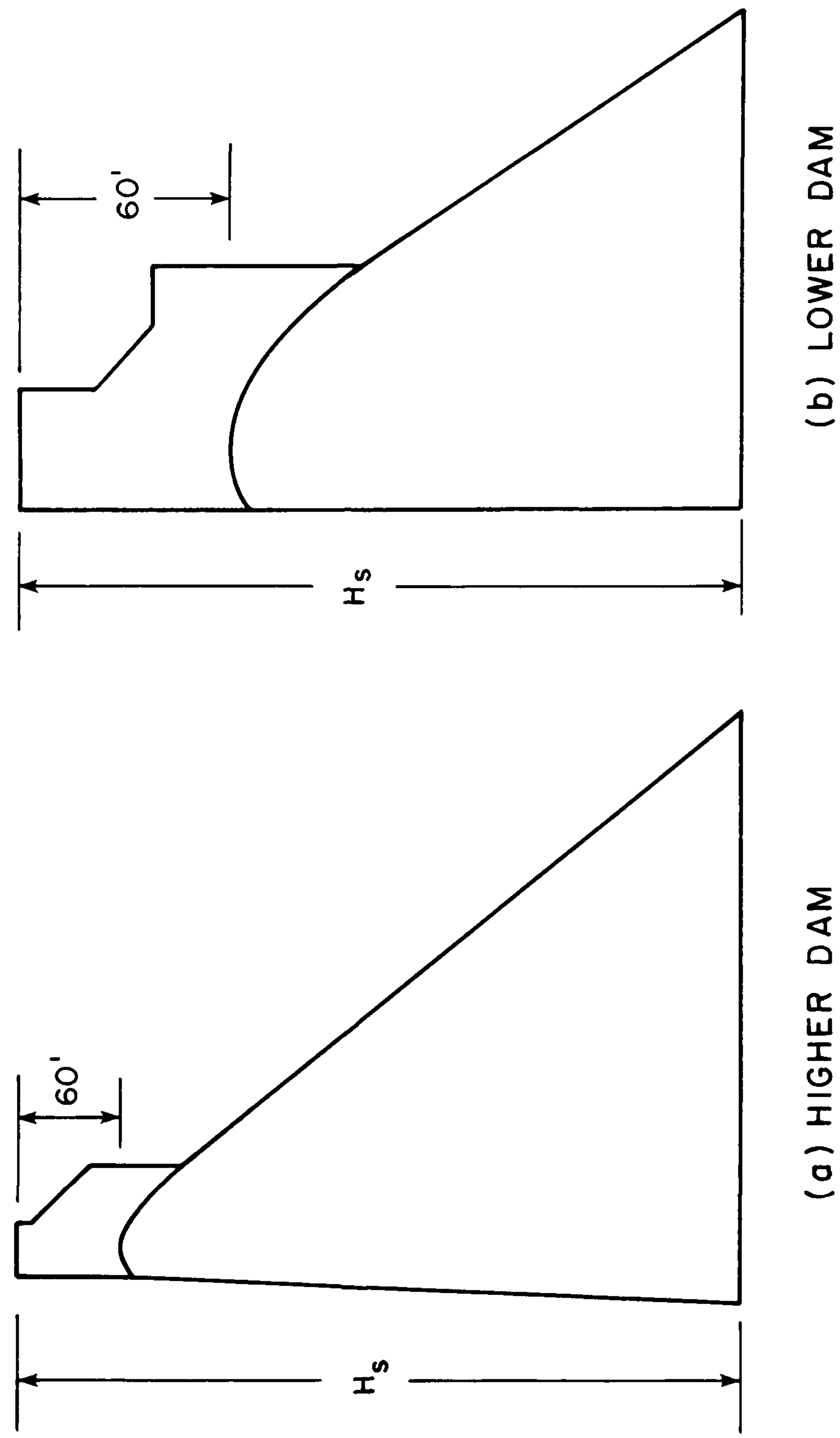

Figure A1 -- Idealized Spillway Monoliths

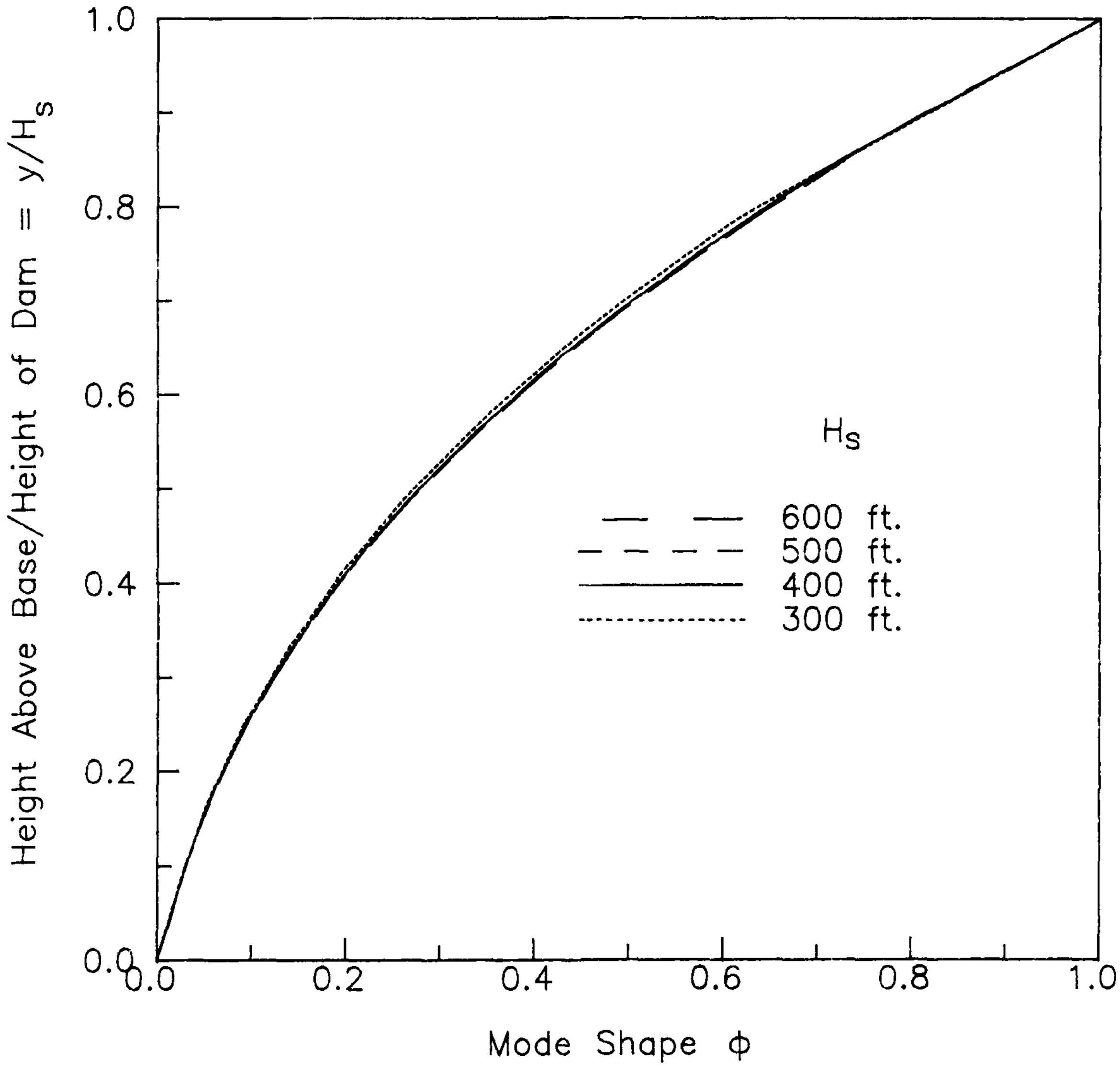

Figure A2 -- Fundamental Mode Shape of Spillway Monolith of Fig. A1(a) for Various Values of H_s

the values of β were determined to be 1.22, 1.21, 1.20, and 1.17 for dams with $H_s = 600$, 500, 400, and 300 ft, respectively. For the dams of Fig. A.1(b) the vibration mode shape is presented in Fig. A.3 and the associated values of β are 1.33, 1.29, 1.23, and 1.17 for $H_s = 300$, 250, 200, and 150 ft. Note that all the mode shapes of Fig. A.2 (or Fig. A.3) and the associated β values would have been identical if the pier height was not fixed and was proportional to H_s.

Based on these results, the standard properties for higher dams ($H_s \geq 300$ ft) are based on the dam of Fig. A.1(a) with $H_s = 400$ ft. Many analyses of this "standard" spillway cross-section were carried out to obtain the standard data presented in this report for higher dams to be used in conjunction with the simplified anlaysis procedure. In particular, the standard value for β is selected to be 1.2 (Eq. 6a) and the standard mode shape is presented in Fig. 3(a) and Table 1(a).

Similarly, the standard properties for lower dams ($H_s < 300$ ft) are based on the dam of Fig. A.1(b) with $H_s = 200$ ft. Many analyses of this "standard" spillway cross-section were carried out to obtain the standard data presented in this report for lower dams. In particular, the standard value for β is selected to be 1.25 (Eq. 6b) and the standard mode shape is presented in Fig. 3(b) and Table 1(b).

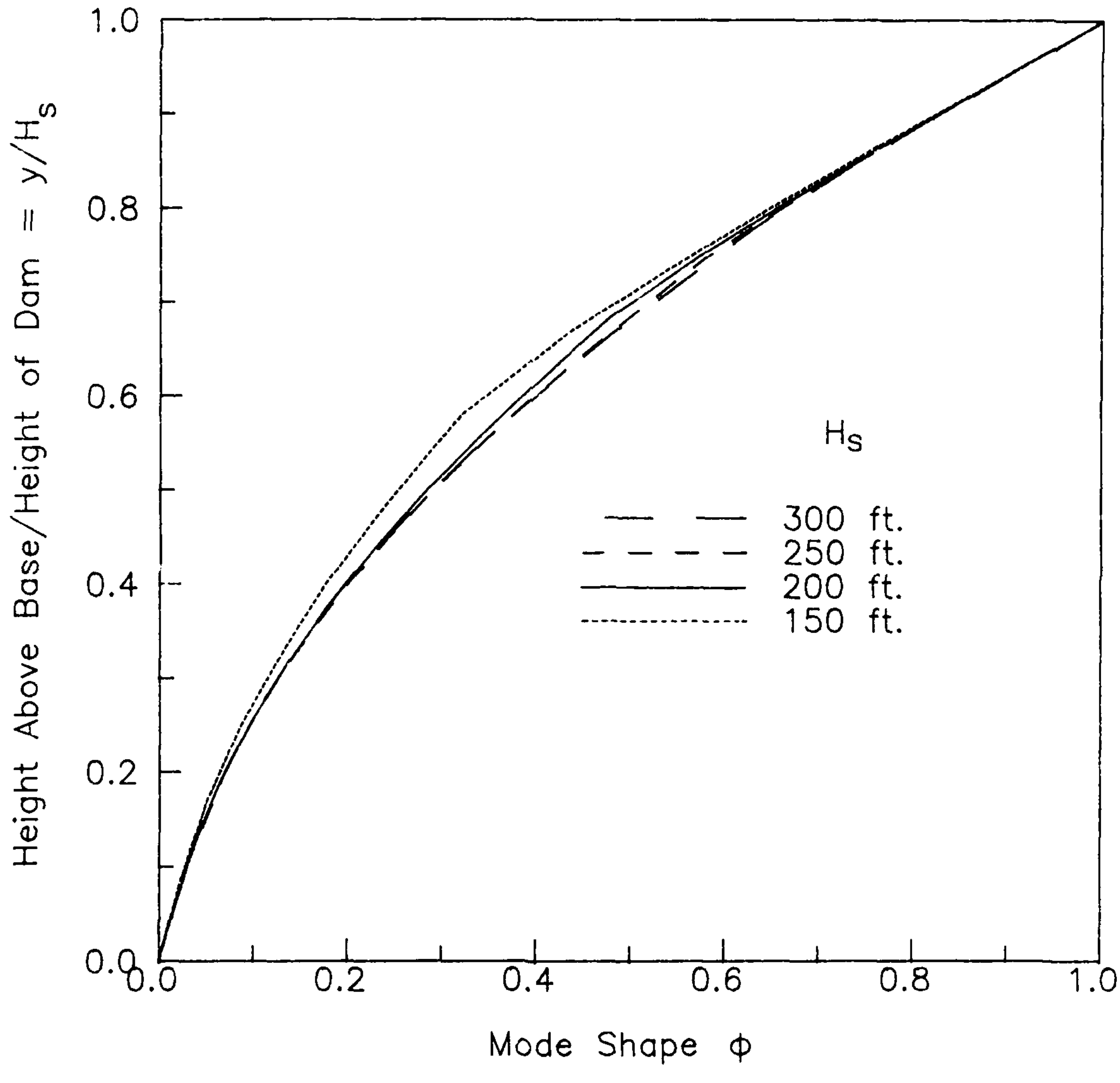

Figure A3 -- Fundamental Mode Shape of Spillway Monolith of Fig. A1(b) for Various Values of H_s

APPENDIX B: DETAILED CALCULATIONS FOR PINE FLAT DAM

This appendix presents the detailed calculations required in the simplified analysis procedure as applied to the tallest, gated spillway monolith of Pine Flat Dam. All computations are performed for the equivalent two-dimensional system of unit thickness representing the dam (see page 3). Only the details for Case 4 in Table 9 (full reservoir and flexible foundation rock) are presented.

Simplified Model of Monolith and Pier

The tallest gated spillway monolith of Pine Flat Dam is divided into fourteen blocks as shown in Fig. 16. Using a unit weight of 155 pcf for the concrete and the ratio of pier width to monolith width = 0.16 in defining $w_s(y)$ for the pier (page 3), the properties of the blocks are presented in Table B.1, from which the total weight is 9434 kips. Replacing the integrals in Eqs. 2b and 3b by the summations over the blocks gives:

$$M_1 \approx \frac{1}{g} \sum_{i=1}^{10} w_i \phi^2(y_i) = \frac{1}{g}(559 \text{ kip}) \tag{B.1}$$

$$L_1 \approx \frac{1}{g} \sum_{i=1}^{10} w_i \phi(y_i) = \frac{1}{g}(1623 \text{ kip}) \tag{B.2}$$

where w_i and y_i are the weight of block i and the elevation of its centroid, respectively. Additional properties of the simplified model are listed in Table B.2.

Equivalent Lateral Forces -- Fundamental Mode

The equivalent lateral earthquake forces $f_1(y)$ are given by Eq. 1, evaluated at each level using $S_a(\tilde{T}_1, \tilde{\xi}_1)/g = 0.377$ (from Table 9 and Fig. 15) and $\tilde{L}_1/\tilde{M}_1 = 3.14$ (from Step 8 in the simplified procedure). The calculations are summarized in Table B.3.

Table B.1 -- Properties of the Simplified Model

Block	Weight w (k)	Elevation of centroid (ft)	ϕ (1) at centroid	$w\phi$ (k)	$w\phi^2$ (k)
14	7.9	394.7	0.976	7.7	7.5
13	34.1	372.6	0.875	29.8	26.1
12	29.3	349.6	0.769	22.5	17.3
11	17.3	337.0	0.713	12.3	8.8
10	197.2	320.7	0.649	127.9	83.0
9	91.1	304.3	0.588	53.6	31.5
8	243.6	289.6	0.537	130.7	70.2
7	641.5	258.9	0.437	280.2	122.4
6	847.4	219.2	0.325	275.4	89.5
5	1053.	179.3	0.231	242.9	56.0
4	1259.	139.5	0.154	194.2	29.9
3	1465.	99.5	0.093	136.8	12.8
2	1671.	59.6	0.048	79.6	3.8
1	1877.	19.6	0.016	29.4	0.5
Total	9434			1623	559

(1) From Fig. 3(a) or Table 1(a).

Table B.2 -- Additional Properties of the Simplified Model

Level	Elevation y (ft)	Width of Monolith $b^{(1)}$ (ft)	Width of Pier b_p (ft)	Weight per Unit Height $w_s^{(2)}$ (k/ft)	Section Modulus $S=1/6\ b^2$ (3) (ft^3)
Top	400.	-	30.0	0.74	-
14	389.38	-	30.0	0.74	-
13	359.286	-	61.3	1.52	-
12	340.	0	61.3	1.52	-
11	335.	29.7	31.6	5.39	-
10	308.85	62.6	0	9.70	653.1
9	300.	70.3	-	10.90	823.7
8	280.	86.9	-	13.47	1259.
7	240.	120.1	-	18.62	2404.
6	200.	153.3	-	23.72	3917.
5	160.	186.5	-	28.91	5797.
4	120.	219.7	-	34.05	8045.
3	80.	252.9	-	39.20	10660.
2	40.	286.1	-	44.35	13640.
1	0.	319.3	-	49.49	16990.

(1) From Fig. 16.

(2) $w_s = 0.155\ b$ for monolith blocks

$= 0.155 \times 0.16\ b_p$ for pier blocks

$= 0.155 \times (\ b + 0.16\ b_p\ )$ for transition blocks

(3) Computed only for monolith blocks, as the pier should be analyzed as a reinforced concrete structure.

Table B.3 -- Equivalent Lateral Earthquake Forces -- Fundamental Vibration Mode

Level	y (ft)	w_s (1) (k/ft)	y/H_s	ϕ (2)	$w_s\phi$ (k/ft)	y/H	(3) gp/wH	gp (k/ft)	(5) $f_1(y)$ (k/ft)
Top	400.	0.74	1.0	1.0	0.74	1.05	0	0	0.87
14	389.38	0.74	0.97	0.95	0.70	1.02	0	0 (4)	0.83
13	359.286	1.52	0.90	0.81	1.23	0.94	0.092	1.98	3.81
12	340.	1.52	0.85	0.73	1.10	0.89	0.144	3.11	4.98
11	335.	5.39	0.84	0.71	3.81	0.88	0.151	3.26	8.36
10	308.85	9.70	0.77	0.61	5.87	0.81	0.178	3.84	11.49
9	300.	10.90	0.75	0.57	6.24	0.79	0.185	3.99	12.11
8	280.	13.47	0.70	0.50	6.79	0.74	0.199	4.29	13.12
7	240.	18.62	0.60	0.38	7.09	0.63	0.209	4.51	13.74
6	200.	23.72	0.50	0.28	6.57	0.53	0.209	4.51	13.12
5	160.	28.91	0.40	0.19	5.55	0.42	0.198	4.27	11.63
4	120.	34.05	0.30	0.12	4.19	0.32	0.189	4.08	9.79
3	80.	39.20	0.20	0.07	2.74	0.21	0.174	3.75	7.69
2	40.	44.35	0.10	0.03	1.33	0.11	0.164	3.54	5.76
1	0.	49.49	0.	0.	0.	0.	0.153	3.30	3.91

(1) From Table B.2

(2) From Fig. 3(a) or Table 1(a)

(3) From step 6, by linearly interpolating the data of Fig. 12 or Table 6

(4) gp = 0 at y = 381 ft, the free surface of water, and varies linearly to 1.98 at level 13

(5) From Eq. 1.

Stress Computation -- Fundamental Mode

The equivalent lateral earthquake forces $f_1(y)$ consist of forces associated with the mass of the dam (the first term of Eq. 1) and the hydrodynamic pressure at the upstream face (the second term). For the purpose of computing bending stresses in the monolith, the forces associated with the mass are applied at the centroids of the blocks. The forces due to the hydrodynamic pressure are applied as a linearly distributed load to the upstream face of each block. Due to these two sets of lateral forces, the resultant bending moments in the monolith are computed at each level from the equations of equilibrium. The normal bending stresses are obtained from elementary beam theory. A computer program (described in Appendix C) was developed for computation of the normal bending stresses in a dam monolith due to equivalent lateral earthquake forces. This is a modified version of the computer program prsented in Ref. [2]. However, an alternative approach in which $f_1(y)$ is computed at the top and bottom of each block is more suitable for hand calculation since it avoids computing the location of the centroid of each block. Using this alternative procedure, the forces $f_1(y)$ and the normal bending stresses σ_{y1} at the two faces of Pine Flat Dam associated with the fundamental vibration mode response of the dam to the earthquake ground motion characterized by the smooth design spectrum of Fig. 15 were computed (Tables B.3 and B.4).

Equivalent Lateral Forces -- Higher Vibration Modes

The equivalent lateral earthquake forces $f_{sc}(y)$ due to the higher vibration modes are given by Eq. 9, evaluated at each level using the maximum ground acceleration for the design earthquake, $a_g = 0.25$ g, and $L_1/M_1 = 2.90$ and $B_1/M_1 = 1.837$. The results are summarized in Table B.5. The calculation of bending moments due to the higher vibration modes is similar to the moment calculations for the fundamental vibration mode, as described previously.

Table B.4 -- Normal Bending Stresses
-- Fundamental Vibration Mode

Level	Section Modulus (1) (ft^3)	Bending Moment (k-ft)	Bending Stress at Faces (psi)
10	653.1	12070.	128.
9	823.7	16500.	139.
8	1259.	30040.	166.
7	2404.	72730.	210.
6	3917.	137100.	243.
5	5797.	222400.	266.
4	8045.	326100.	281.
3	10660.	445400.	290.
2	13640.	577000.	294.
1	16990.	717900.	293.

(1) From Table B.2.

Table B.5 -- Equivalent Lateral Earthquake Forces -- Higher Vibration Modes

Level	y (ft)	(1) $w_s[1-\frac{L_1}{M_1}\phi]$ (k/ft)	y/H	(2) $\frac{gp_o}{wH}$	gp_o (k/ft)	(3) $[gp_o-\frac{B_1}{M_1}w_s\phi]$ (k/ft)	(4) $f_{sc}(y)$ (k/ft)
Top	400.	-1.41	1.05	0.	0.	-1.36	-0.69
14	389.38	-1.30	1.02	0.	0.	-1.29	-0.65
13	359.286	-2.06	0.94	0.15	3.54	1.28	-0.20
12	340.	-1.68	0.89	0.24	5.61	3.59	0.48
11	335.	-5.66	0.88	0.26	6.09	-0.90	-1.64
10	308.85	-7.34	0.81	0.35	8.30	-2.48	-2.46
9	300.	-7.20	0.79	0.38	8.96	-2.49	-2.42
8	280.	-6.24	0.74	0.43	10.27	-2.20	-2.11
7	240.	-1.98	0.63	0.52	12.46	-0.57	-0.64
6	200.	4.64	0.53	0.60	14.15	2.08	1.68
5	160.	12.79	0.42	0.65	15.45	5.26	4.51
4	120.	21.89	0.32	0.69	16.43	8.74	7.66
3	80.	31.23	0.21	0.72	17.12	12.08	10.83
2	40.	40.49	0.11	0.74	17.50	15.05	13.89
1	0.	49.49	0.	0.74	17.64	17.64	16.78

(1) w_s and ϕ from Table B.3

(2) From linear interpolation of data from Fig. 13 or Table 8

(3) $w_s\phi$ from Table B.3; $gp_o = 0$ at $y = 381$ ft, the free surface of water, and varies linearly to 3.54 at level 13

(4) From Eq. 9.

Stress Computation -- Higher Modes

The normal bending stresses at the faces of the monolith due to the equivalent lateral earthquake forces $f_{sc}(y)$ are computed by the procedure described above for stresses due to forces $f_1(y)$. The resulting normal bending stresses $\sigma_{y,sc}$ presented in Table B.6 are due to the response contributions of the higher vibration modes.

Table B.6 -- Normal Bending Stresses
-- Higher Vibration Modes

Level	Section Modulus[(1)] (ft^3)	Bending Moment (k-ft)	Bending Stress at Faces (psi)
10	653.1	-2382.	-25.
9	823.7	-3181.	-27.
8	1259.	-5649.	-31.
7	2404.	-12600.	-36.
6	3917.	-20410.	-36.
5	5797.	-25440.	-30.
4	8045.	-23160.	-20.
3	10660.	-8605.	-6.
2	13640.	23290.	12.
1	16990.	77300.	32.

(1) From Table B.2.

APPENDIX C: COMPUTER PROGRAM FOR STRESS COMPUTATION

This appendix describes a computer program for computing the stresses in a nonoverflow or a spillway monolith of a concrete gravity dam using the results of the step-by-step simplified analysis procedure presented in this report. The program computes the bending stresses due to the equivalent lateral forces, $f_1(y)$ and $f_{sc}(y)$, representing the maximum effects of the fundamental and higher vibration modes of the dam, respectively. The program also computes the direct and bending stresses due to the self-weight of the dam and hydrostatic pressure. Transformation to principal stresses and combination of stresses due to the three load cases are not performed.

The program is written in FORTRAN 77 for interactive execution.

Simplified Model of Dam Monolith

A dam monolith is modeled as a series of blocks, numbered sequentially from the base to the crest. Increasing the number of blocks increases the accuracy of the computed stresses. The free surface of the impounded water may be at any elevation. The elevation of the reservoir bottom must be equal to the elevation of a block bottom. Fig. C.1 shows the features of the simplified block model.

Program Input

The program queries the user for all input data, which are entered free-format. The program assumes that the unit of length is feet, the unit of force and weight is kips, the unit of acceleration is g's, and the unit of stress is psi. The input data are as follows:

1. ITYPE, to identify the type of dam monolith. ITYPE = 1 for non-overflow monolith; ITYPE = 2 for spillway monolith.
2. N, the total number of blocks in the simplified model.

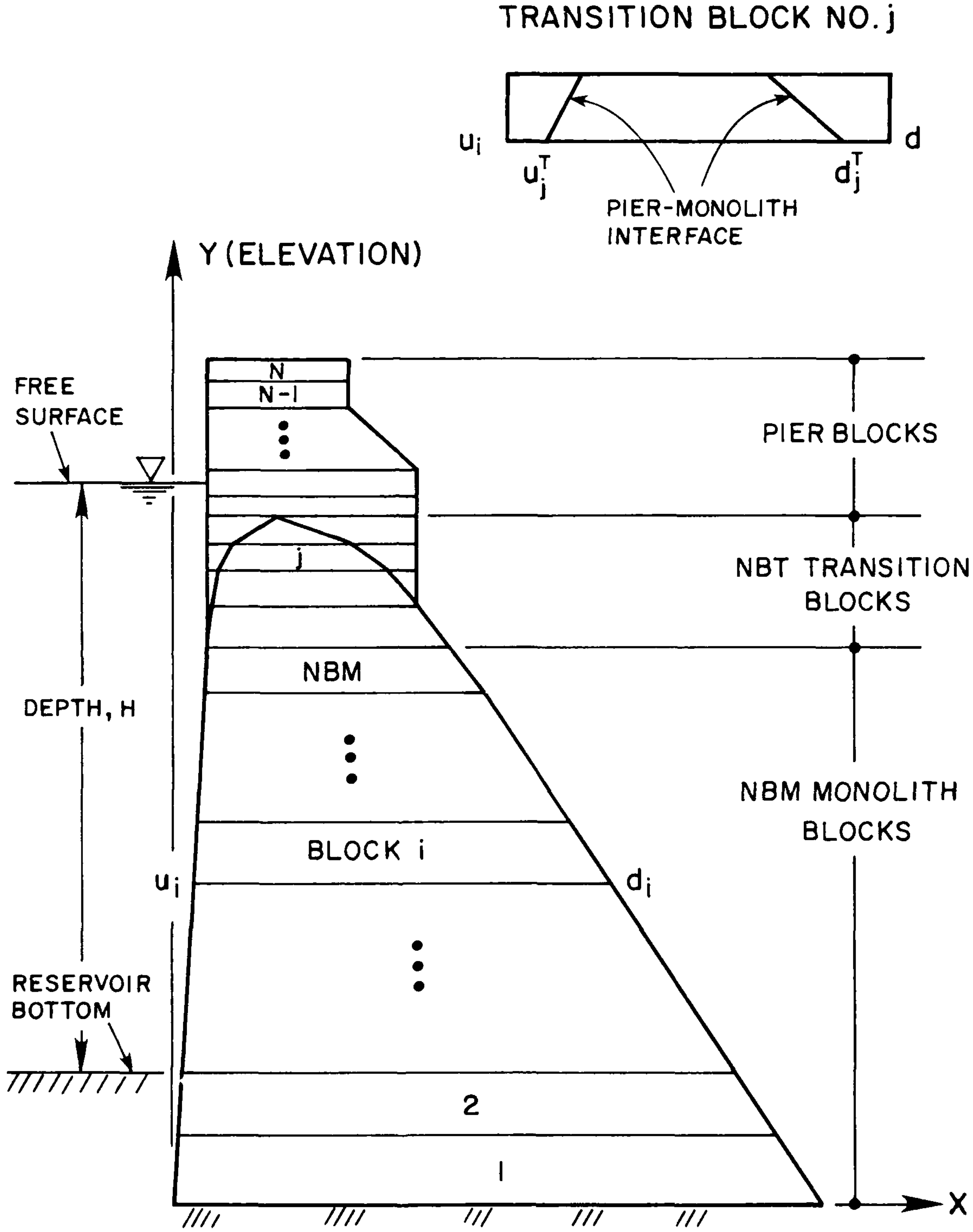

Figure C1 -- Block Model of Spillway Section

3. NBM, the number of blocks in the monolith portion of spillway section. This input is queried only for ITYPE = 2.
4. NBT, the number of transition blocks containing parts from both monolith and pier of the spillway section. This input is queried only for ITYPE = 2.
5. The ratio of the width of pier to the width of monolith, queried only for ITYPE = 2.
6. The default unit weight of concrete in the dam.
7. For the bottom of each block i, the x-coordinate u_i at the upstream face, the x-coordinate d_i at the downstream face, the elevation, and the unit weight of concrete in the block (enter zero if default unit weight).
8. For block j = NBM+1 to NBM+NBT+1, the x-coordinates u_j^T and d_j^T of the upstream and downstream interfaces of the monolith and the pier of the spillway section, respectively, queried only for ITYPE = 2.
9. The x-coordinates of the upstream and downstream faces and the elevation of the dam crest.
10. An alternate value for the ratio L_1/M_1, if desired, where M_1 and L_1 are the generalized mass and earthquake force coefficient for the dam on rigid foundation rock with empty reservoir (Eqs. 2b and 3b). If not specified, the value of L_1/M_1 computed from the block model (as in Steps 7 and 8 of the step-by-step procedure) is used.

The remaining data are entered for each case:

11. The elevations of the free surface of water and reservoir bottom.
12. The ordinates of the hydrodynamic pressure function, gp/wH, at the y/H values indicated. The ordinates are obtained from Step 6 of the step-by-step procedure.
13. The pseudo-acceleration ordinate of the earthquake design spectrum evaluated at the fundamental vibration period and damping ratio of the dam as evaluated in Step 9 of the step-by-step procedure.

14. The ratio $\tilde{L}_1/\tilde{M}_1$, where $\tilde{M}_1$ and $\tilde{L}_1$ = the generalized mass and earthquake force coefficient, including hydrodynamic effects determined in Steps 7 and 8 of the step-by-step procedure. This ratio reduces to L_1/M_1 for a dam with empty reservoir.
15. An alternate value of B_1/M_1, if desired. If not specified, the value computed in Step 11 of the step-by-step procedure is used.
16. The maximum ground acceleration of the design earthquake.

Computed Response

The program computes the vertical, normal (bending) stresses at the bottom of each block of the monolith at the upstream and downstream faces based on simple beam theory. The stresses are not computed in the pier portion of an overflow section. Stresses are computed for three loading cases: (1) static forces (self-weight of the dam and hydrostatic pressure); (2) equivalent lateral forces associated with the fundamental vibration mode; and (3) the equivalent lateral forces associated with the higher vibration modes. The unit of stress is pounds per square inch.

Example

The use of the computer program in the stress computation for the spillway section of Pine Flat Dam is illustrated in the listing shown next wherein the computed vertical, normal (bending) stresses due to the four loading cases are also presented.

```
ENTER DAM TYPE
( 1 = NON-OVERFLOW SECTION, 2 = OVERFLOW SECTION ):2

ENTER THE NUMBER OF BLOCKS IN THE DAM: 14

     ENTER NO. OF BLOCKS OF MONOLITH: 9

     ENTER NO. OF TRANSITION BLOCKS: 2

     ENTER WIDTH RATIO OF PIER AND MONOLITH: .16

ENTER THE DEFAULT UNIT WEIGHT: .155

     ENTER X1,X2,Y, AND UNIT WEIGHT OF BLOCK NO.  1: 0,319.283,0,0

     ENTER X1,X2,Y, AND UNIT WEIGHT OF BLOCK NO.  2: 2,288.082,40,0

     ENTER X1,X2,Y, AND UNIT WEIGHT OF BLOCK NO.  3: 4,256.881,80,0

     ENTER X1,X2,Y, AND UNIT WEIGHT OF BLOCK NO.  4: 6,225.679,120,0

     ENTER X1,X2,Y, AND UNIT WEIGHT OF BLOCK NO.  5: 8,194.478,160,0

     ENTER X1,X2,Y, AND UNIT WEIGHT OF BLOCK NO.  6: 10,163.277,200,0

     ENTER X1,X2,Y, AND UNIT WEIGHT OF BLOCK NO.  7: 12,132.076,240,0

     ENTER X1,X2,Y, AND UNIT WEIGHT OF BLOCK NO.  8: 14,100.874,280,0

     ENTER X1,X2,Y, AND UNIT WEIGHT OF BLOCK NO.  9: 15,85.274,300,0

     ENTER X1,X2,Y, AND UNIT WEIGHT OF BLOCK NO. 10: 15.4425,78,308.85,0

     ENTER X1 AND X2 OF TRANSITION LEVEL: 15.4425,78

     ENTER X1,X2,Y, AND UNIT WEIGHT OF BLOCK NO. 11: 16.75,73,335,0

     ENTER X1 AND X2 OF TRANSITION LEVEL: 16.75,46.434
```

ENTER X1 AND X2 OF TRANSITION LEVEL: 27.75,27.75

ENTER X1,X2,Y, AND UNIT WEIGHT OF BLOCK NO. 13: 16.75,78,359.286,0

ENTER X1,X2,Y, AND UNIT WEIGHT OF BLOCK NO. 14: 16.75,46.75,389.380,0

ENTER X1,X2, AND Y AT THE CREST: 16.75,46.75,400

CHECK INPUT DATA :
NBLOCK = 14 NBM = 9 NBT = 2

BLOCK	XLEFT	XTRAN	XTRAN	XRIGHT	Y
	16.750			46.750	400.000
14					
	16.750	16.750	46.750	46.750	389.380
13					
	16.750	16.750	78.000	78.000	359.286
12					
	16.750	27.750	27.750	78.000	340.000
11					
	16.750	16.750	46.434	78.000	335.000
10					
	15.443	15.443	78.000	78.000	308.850
9					
	15.000	15.000	85.274	85.274	300.000
8					
	14.000	14.000	100.874	100.874	280.000
7					
	12.000	12.000	132.076	132.076	240.000
6					
	10.000	10.000	163.277	163.277	200.000
5					
	8.000	8.000	194.478	194.478	160.000
4					
	6.000	6.000	225.679	225.679	120.000
3					
	4.000	4.000	256.881	256.881	80.000
2					
	2.000	2.000	288.082	288.082	40.000
1					
	0.000	0.000	319.283	319.283	0.000

PROPERTIES OF THE DAM

LOCK	CENTROID ELEV.	WEIGHT
14	394.690	7.901
13	372.615	34.051
12	349.643	29.295
11	337.033	17.257
10	320.679	197.174
9	304.339	91.106
8	289.648	243.579
7	258.930	641.545

6	219.190	847.394
5	179.349	1053.240
4	139.455	1259.087
3	99.532	1464.936
2	59.589	1670.785
1	19.634	1876.631

		9433.984

FUNDAMENTAL VIBRATION PROPERTIES OF THE DAM

L1 = 1623.176 M1 = 559.265

THE FACTOR L1/M1 = 2.902

ENTER AN ALTERNATE VALUE FOR L1/M1:0

DO YOU WANT TO CONTINUE? (0=YES,1=NO):0

ENTER ELEVATION OF FREE-SURFACE: 0

ENTER ELEVATION OF RESERVOIR BOTTOM:0

STATIC STRESSES IN DAM

BLOCK	UPSTREAM FACE	DOWNSTREAM FACE
10	-47.681	-15.745
9	-59.408	-15.059
8	-84.021	-15.159
7	-129.991	-15.971
6	-173.756	-17.375
5	-216.414	-19.133
4	-258.440	-21.111
3	-300.073	-23.232
2	-341.444	-25.455
1	-382.632	-27.749

ENTER THE PSUEDO-ACCELERATION ORDINATE IN G: .677

ENTER L1(TILDE)/M1(TILDE) FACTOR: 2.90

FUNDAMENTAL MODE STRESSES IN DAM

BLOCK	UPSTREAM FACE	DOWNSTREAM FACE
10	111.618	-111.618
9	121.655	-121.655
8	148.300	-148.300

7	195.001	-195.001
6	230.668	-230.668
5	255.373	-255.373
4	270.639	-270.639
3	278.129	-278.129
2	[illegible]	[illegible]
1	275.944	-275.944

THE FACTOR B1/M1 IS = 0.000
ENTER AN ALTERNATE VALUE FOR B1/M1:0

ENTER MAX. GROUND ACCELERATION IN G: .25

HIGHER MODE STRESSES IN DAM

BLOCK	UPSTREAM FACE	DOWNSTREAM FACE
10	-22.983	22.983
9	-24.317	24.317
8	-27.655	27.655
7	-31.153	31.153
6	-30.549	30.549
5	-26.045	26.045
4	-18.189	18.189
3	-7.564	7.564
2	5.276	-5.276
1	19.813	-19.813

DO YOU WANT TO CONTINUE? (0=YES,1=NO):0

ENTER ELEVATION OF FREE-SURFACE: 381

ENTER ELEVATION OF RESERVOIR BOTTOM:0

STATIC STRESSES IN DAM

BLOCK	UPSTREAM FACE	DOWNSTREAM FACE
10	-8.202	-56.294
9	-15.470	-60.368
8	-28.813	-72.384
7	-51.994	-97.173
6	-72.818	-122.644
5	-92.466	-148.509
4	-111.445	-174.616
3	-130.008	-200.879
2	-143.294	-227.252
1	-166.386	-253.702

ENTER THE HYDRODYNAMIC PRESSURE FOR THE
FUNDAMENTAL VIBRATION MODE OF THE DAM

ENTER THE PRESSURE ORDINATE FOR Y/H =0.943: .092

ENTER THE PRESSURE ORDINATE FOR Y/H =0.892: .144

ENTER THE PRESSURE ORDINATE FOR Y/H =0.879: .151

ENTER THE PRESSURE ORDINATE FOR Y/H =0.811: .178

ENTER THE PRESSURE ORDINATE FOR Y/H =0.787: .185

ENTER THE PRESSURE ORDINATE FOR Y/H =0.735: .199

ENTER THE PRESSURE ORDINATE FOR Y/H =0.630: .209

ENTER THE PRESSURE ORDINATE FOR Y/H =0.525: .209

ENTER THE PRESSURE ORDINATE FOR Y/H =0.420: .198

ENTER THE PRESSURE ORDINATE FOR Y/H =0.315: .189

ENTER THE PRESSURE ORDINATE FOR Y/H =0.210: .174

ENTER THE PRESSURE ORDINATE FOR Y/H =0.105: .164

ENTER THE PRESSURE ORDINATE FOR Y/H =0.000: .153

ENTER THE PSUEDO-ACCELERATION ORDINATE IN G: .542

ENTER L1(TILDE)/M1(TILDE) FACTOR: 3.14

FUNDAMENTAL MODE STRESSES IN DAM

BLOCK	UPSTREAM FACE	DOWNSTREAM FACE
10	184.558	-184.558
9	200.011	-200.011
8	238.321	-238.321
7	302.053	-302.053
6	349.554	-349.554
5	382.941	-382.941
4	404.679	-404.679
3	417.126	-417.126
2	422.237	-422.237
1	421.778	-421.778

THE FACTOR B1/M1 IS = 1.837
ENTER AN ALTERNATE VALUE FOR B1/M1:0

ENTER MAX. GROUND ACCELERATION IN G: .25

HIGHER MODE STRESSES IN DAM

BLOCK	UPSTREAM FACE	DOWNSTREAM FACE
10	-25.332	25.332
9	-26.820	26.820
8	-31.167	31.167
7	-36.386	36.386
6	-36.179	36.179
5	-30.472	30.472
4	-19.995	19.995
3	-5.606	5.606
2	11.857	-11.857
1	31.593	-31.593

DO YOU WANT TO CONTINUE? (O=YES,1=NO):0

ENTER ELEVATION OF FREE-SURFACE: 0

ENTER ELEVATION OF RESERVOIR BOTTOM:0

STATIC STRESSES IN DAM

BLOCK	UPSTREAM FACE	DOWNSTREAM FACE
10	-47.681	-15.745
9	-59.408	-15.059
8	-84.021	-15.159
7	-129.991	-15.971
6	-173.756	-17.375
5	-216.414	-19.133
4	-258.440	-21.111
3	-300.073	-23.232
2	-341.444	-25.455
1	-382.632	-27.749

ENTER THE PSUEDO-ACCELERATION ORDINATE IN G: .453

ENTER L1(TILDE)/M1(TILDE) FACTOR: 2.90

FUNDAMENTAL MODE STRESSES IN DAM

BLOCK	UPSTREAM FACE	DOWNSTREAM FACE
10	74.686	-74.686
9	81.403	-81.403
8	99.232	-99.232
7	130.481	-130.481
6	154.346	-154.346
5	170.877	-170.877
4	181.092	-181.092
3	186.104	-186.104
2	186.963	-186.963
1	184.642	-184.642

THE FACTOR B1/M1 IS = 0.000
ENTER AN ALTERNATE VALUE FOR B1/M1:0

ENTER MAX. GROUND ACCELERATION IN G: .25

HIGHER MODE STRESSES IN DAM

BLOCK	UPSTREAM FACE	DOWNSTREAM FACE
10	-22.983	22.983
9	-24.317	24.317
8	-27.655	27.655
7	-31.153	31.153
6	-30.549	30.549
5	-26.045	26.045
4	-18.189	18.189
3	-7.564	7.564
2	5.276	-5.276
1	19.813	-19.813

DO YOU WANT TO CONTINUE? (0=YES,1=NO):0

ENTER ELEVATION OF FREE-SURFACE: 381

ENTER ELEVATION OF RESERVOIR BOTTOM:0

STATIC STRESSES IN DAM

BLOCK	UPSTREAM FACE	DOWNSTREAM FACE
10	-8.202	-56.294
9	-15.470	-60.368
8	-28.813	-72.384

7	-51.994	-97.173
6	-72.818	-122.644
5	-92.466	-148.509
4	-111.445	-174.616
3	-130.008	-200.879
2	-148.294	-227.252
1	-166.386	-253.702

ENTER THE HYDRODYNAMIC PRESSURE FOR THE
FUNDAMENTAL VIBRATION MODE OF THE DAM

ENTER THE PRESSURE ORDINATE FOR Y/H =0.943: .092

ENTER THE PRESSURE ORDINATE FOR Y/H =0.892: .144

ENTER THE PRESSURE ORDINATE FOR Y/H =0.879: .151

ENTER THE PRESSURE ORDINATE FOR Y/H =0.811: .178

ENTER THE PRESSURE ORDINATE FOR Y/H =0.787: .185

ENTER THE PRESSURE ORDINATE FOR Y/H =0.735: .199

ENTER THE PRESSURE ORDINATE FOR Y/H =0.630: .209

ENTER THE PRESSURE ORDINATE FOR Y/H =0.525: .209

ENTER THE PRESSURE ORDINATE FOR Y/H =0.420: .198

ENTER THE PRESSURE ORDINATE FOR Y/H =0.315: .189

ENTER THE PRESSURE ORDINATE FOR Y/H =0.210: .174

ENTER THE PRESSURE ORDINATE FOR Y/H =0.105: .164

ENTER THE PRESSURE ORDINATE FOR Y/H =0.000: .153

ENTER THE PSUEDO-ACCELERATION ORDINATE IN G: .377

ENTER L1(TILDE)/M1(TILDE) FACTOR: 3.14

FUNDAMENTAL MODE STRESSES IN DAM

BLOCK	UPSTREAM FACE	DOWNSTREAM FACE
10	128.373	-128.373

9	139.122	-139.122
8	165.769	-165.769
7	210.099	-210.099
6	243.140	-243.140
5	266.363	-266.363
4	281.483	-281.483
3	290.141	-290.141
2	293.696	-293.696
1	293.377	-293.377

THE FACTOR B1/M1 IS = 1.837
ENTER AN ALTERNATE VALUE FOR B1/M1:0

ENTER MAX. GROUND ACCELERATION IN G: .25

HIGHER MODE STRESSES IN DAM

BLOCK	UPSTREAM FACE	DOWNSTREAM FACE
10	-25.332	25.332
9	-26.820	26.820
8	-31.167	31.167
7	-36.386	36.386
6	-36.179	36.179
5	-30.472	30.472
4	-19.995	19.995
3	-5.606	5.606
2	11.857	-11.857
1	31.593	-31.593

DO YOU WANT TO CONTINUE? (0=YES,1=NO):1

```
C*********************************************************************
C
C         A COMPUTER PROGRAM TO PERFORM A SIMPLIFIED ANALYSIS
C               OF CONCRETE GRAVITY DAMS DUE TO EARTHQUAKES
C             INCLUDING THE EFFECTS OF DAM-WATER INTERACTION,
C    DAM-FOUNDATION ROCK INTERACTION, AND RESERVOIR BOTTOM ABSORPTION
C
C                           HANCHEN TAN
C             THE UNIVERSITY OF CALIFIRNIA AT BERKELEY
C
C                          VERSION 2.0 :
C         A MODIFICATION OF VERSION 1.0, JANUARY 1985 BY
C                         GREGORY FENVES
C
C                          NOVEMBER 1987
C
C*********************************************************************
C
      CALL SIMPL
      STOP
      END
C--------------------------------------------------------------------
      SUBROUTINE SIMPL
C
C         MAIN SUBPROGRAM -- CONTROL THE EXECUTION OF THE PROGRAM
C
      DIMENSION BLOCKS(5,21),PRESS(21),WEIGHT(20),STRSTA(2,20),
     1          STRWGT(2,20),STRDUM(2,20),STRFUN(2,20),
     2          STRDYN(2,20),STRCOR(2,20),STRWCR(2,20)
C
      DIMENSION PARTFC(3)
      CHARACTER*40 TITSTA,TITFUN,TITCOR
C
      DATA TITSTA/'            STATIC STRESSES IN DAM       '/,
     1     TITFUN/'     FUNDAMENTAL MODE STRESSES IN DAM    '/,
     2     TITCOR/'        HIGHER MODE STRESSES IN DAM      '/
C
C         THESE DATA STATEMENTS, AND THE FIRST DIMENSION STATEMENT,
C         DETERMINE THE MAXIMUM NUMBER OF BLOCKS THAT MAY BE USED
C
      DATA NMAX/20/
C
C         THIS DATA STATEMENT CONTAINS UNIT-DEPENDENT CONSTANTS
C
      DATA GAMMA/0.0624/,STRCON/0.144/
C
C         READ THE PROPERTIES OF THE BLOCKS, COMPUTE OTHER BLOCK
C         PROPERTIES, AND COMPUTE THE STATIC STRESSES DUE TO THE
C         WEIGHT AND EFFECTIVE EARTHQUAKE FORCE
C
      CALL DAMPRP (BLOCKS,NBLOCK,NBM,NBT,ITYPE,NMAX,WEIGHT,STRWGT,
     1            STRWCR,STRFUN,PARTFC)
C
C         TEST TO CONTINUE WITH EXECUTION OF PROGRAM
C
```

```
   10 WRITE (*,99)
      READ (*,*) I
      IF (I.NE.0) GO TO 20
C
C          READ IN PROPERTIES OF THE IMPOUNDED WATER
C          AND COMPUTE STRESSES DUE TO HYDROSTATIC PRESSURE
C
      CALL REDWAT (H,HB)
      CALL DAMSTA (BLOCKS,NBLOCK,GAMMA,H,HB,PRESS,STRDUM)
      STRFAC = 1.0/STRCON
      NB = NBLOCK
      IF (ITYPE.EQ.2) NB = NBM + 1
      CALL COMSTA (NB,STRWGT,STRDUM,STRSTA,STRFAC,TITSTA)
C
C          COMPUTE THE DYNAMIC STRESSES DUE TO THE FUNDAMENTAL
C          MODE SHAPE
C
      CALL DAMDYN (BLOCKS,NBLOCK,GAMMA,H,HB,PRESS,STRDUM)
      WRITE (*,97)
      READ (*,*) SA
      STRFAC = SA/STRCON
      WRITE (*,95)
      READ (*,*) SA
      STRFAC = STRFAC*SA
      CALL COMSTA (NB,STRFUN,STRDUM,STRDYN,STRFAC,TITFUN)
C
C          COMPUTE THE HIGHER MODE STRESSES
C
      CALL DAMCOR (BLOCKS,NBLOCK,ITYPE,GAMMA,H,HB,PARTFC(2),STRFUN,
     1                  PRESS,STRDUM)
      WRITE (*,94)
      READ (*,*) SA
      STRFAC = SA/STRCON
      CALL COMSTA (NB,STRWCR,STRDUM,STRCOR,STRFAC,TITCOR)
      GO TO 10
C
   20 RETURN
C
   99 FORMAT (/////' DO YOU WANT TO CONTINUE? (0=YES,1=NO):')
   97 FORMAT (//' ENTER THE PSUEDO-ACCELERATION ORDINATE IN G: ')
   95 FORMAT (//' ENTER L1(TILDE)/M1(TILDE) FACTOR: ')
   94 FORMAT (//' ENTER MAX. GROUND ACCELERATION IN G: ')
C
      END
C-----------------------------------------------------------------------
      SUBROUTINE DAMPRP (BLOCKS,NBLOCK,NBM,NBT,ITYPE,NMAX,WEIGHT,
     1                   STRWGT,STRWCR,STRFUN,PARTFC)
C
C          INPUT THE PROPERTIES OF THE DAM, AND COMPUTE THE STATIC
C          AND FUNDAMENTAL VIBRATION PROPERTIES OF THE DAM
C
      DIMENSION BLOCKS(5,1),WEIGHT(1),STRWGT(2,1),
     1          STRWCR(2,1),STRFUN(2,1),PARTFC(3)
      DIMENSION TRANS(2,21)
      EXTERNAL VALWGT,VALHOR
```

```
C
C          INPUT BLOCK PROPERTIES AND COMPUTE STATIC STRESSES
C
      WRITE (*,100)
      READ  (*,*) ITYPE
      IF ( ITYPE.NE.1 .AND. ITYPE.NE.2 ) STOP
C
      CALL REDBLK (BLOCKS,TRANS,NBLOCK,NBM,NBT,ITYPE,NMAX,
     1             WIDTHR,WEIGHT)
      CALL BLCKVL (BLOCKS,TRANS,NBLOCK,NBM,NBT,ITYPE,WIDTHR,
     1             WEIGHT,WEIGHT)
      CALL STRLOD (BLOCKS,NBLOCK,WEIGHT,VALWGT,STRWGT)
      CALL STRLOD (BLOCKS,NBLOCK,WEIGHT,VALHOR,STRWCR)
C
      WRITE (*,99)
      DO 10 J = 1,NBLOCK
          I = NBLOCK + 1 - J
          WRITE (*,98) I, BLOCKS(5,I), WEIGHT(I)
   10     CONTINUE
C
C         COMPUTE THE FUNDAMENTAL VIBRATION PROPERTIES AND STRESSES
C         DUE TO THE EFFECTIVE EARTHQUAKE FORCE
C
      CALL FUNMOD (BLOCKS,NBLOCK,ITYPE,WEIGHT,WEIGHT,XTOT,PARTFC(1),
     1             PARTFC(2))
      CALL STRLOD (BLOCKS,NBLOCK,WEIGHT,VALHOR,STRFUN)
C
      PARTFC(3) = PARTFC(1)/PARTFC(2)
      WRITE (*,97) XTOT,(PARTFC(I),I=1,3)
      READ (*,*) DUM
      IF (DUM.GT.0.0) PARTFC(3) = DUM
C
C         COMPUTE THE HIGHER MODE STRESSES DUE TO THE
C         WEIGHT OF THE DAM
C
      DO 20 J = 1,NBLOCK
          STRWCR(1,J) = STRWCR(1,J) - PARTFC(3)*STRFUN(1,J)
          STRWCR(2,J) = STRWCR(2,J) - PARTFC(3)*STRFUN(2,J)
   20     CONTINUE
C
      RETURN
C
  100 FORMAT (//3X,'ENTER DAM TYPE'//3X,'( 1 = NON-OVERFLOW SECTION,'
     1         ,' 2 = OVERFLOW SECTION ):')
   99 FORMAT (//3X,'PROPERTIES OF THE DAM'//'BLOCK',2X,'CENTROID',
     1         4X,'WEIGHT'/10X,'ELEV.'/1X,27('-')/)
   98 FORMAT (3X,I2,3X,F8.3,3X,F8.3)
   97 FORMAT (19X,'--------'/19X,F8.3//
     1         ' FUNDAMENTAL VIBRATION PROPERTIES OF THE DAM'//
     2         5X,' L1 =',F9.3,5X,' M1 =',F9.3//
     3         5X,' THE FACTOR L1/M1 =',F9.3//
     4         ' ENTER AN ALTERNATE VALUE FOR L1/M1:')
C
      END
C-------------------------------------------------------------------
```

```
      SUBROUTINE DAMCOR (BLOCKS,NBLOCK,ITYPE,GAMMA,H,HB,XM1,STRFUN,
     1                        PRESS,STRDUM)
C
      DIMENSION BLOCKS(5,1),PRESS(1),STRFUN(2,1),STRDUM(2,1)
      EXTERNAL VALHDY
      DATA BFACT1/0.20/,BFACT2/0.25/
C
      CALL CORPRS (BLOCKS,NBLOCK,H,HB,GAMMA,PRESS)
      CALL STRPRS (BLOCKS,NBLOCK,PRESS,1,H,HB,VALHDY,STRDUM)
C
      D = H - HB
      HS = BLOCKS(3,NBLOCK+1) - BLOCKS(3,1)
      BFACT = BFACT1
      IF (ITYPE.EQ.2) BFACT = BFACT2
      B01M1 = 0.5*BFACT*GAMMA*D*D*D*D/(HS*HS*XM1)
      WRITE (*,99) B01M1
      READ (*,*) D
      IF (D.GT.0.0) B01M1 = D
C
      DO 10 J = 1,NBLOCK
          STRDUM(1,J) = STRDUM(1,J) - B01M1*STRFUN(1,J)
          STRDUM(2,J) = STRDUM(2,J) - B01M1*STRFUN(2,J)
   10     CONTINUE
C
      RETURN
C
   99 FORMAT (//' THE FACTOR B1/M1 IS = ',F9.3/
     1            ' ENTER AN ALTERNATE VALUE FOR B1/M1:')
      END
C-----------------------------------------------------------------------
      SUBROUTINE REDBLK (BLOCKS,TRANS,NBLOCK,NBM,NBT,ITYPE,NMAX,
     1                        WIDTHR,UNITWT)
C
C         READ THE PROPERTIES OF THE BLOCKS IN THE DAM
C
      DIMENSION BLOCKS(5,1),TRANS(2,1),UNITWT(1)
C
      WRITE (*,99)
      READ (*,*) NBLOCK
      IF (NBLOCK.GT.NMAX) GO TO 20
      NBM = NBLOCK
      NBT = 0
      WIDTHR = 1.
      IF (ITYPE.EQ.1) GO TO 5
      WRITE (*,100)
      READ (*,*) NBM
      WRITE (*,101)
      READ (*,*) NBT
          NBMT = NBM + NBT
          WRITE (*,102)
          READ (*,*) WIDTHR
C
    5 WRITE (*,97)
      READ (*,*) DEFWGT
C
```

```
      DO 10 I = 1,NBLOCK
           WRITE (*,95) I
           READ (*,*) (BLOCKS(J,I),J=1,3),UNT
           TRANS(1,I) = BLOCKS(1,I)
           TRANS(2,I) = BLOCKS(2,I)
           IF (I.LE.NBM .OR. I.GT.NBMT+1) GO TO 6
           WRITE (*,103)
           READ (*,*) (TRANS(J,I),J=1,2)
    6      IF (UNT.LE.0.0) UNT = DEFWGT
           UNITWT(I) = UNT
   10      CONTINUE
C
      NBL1 = NBLOCK + 1
      WRITE (*,93)
      READ (*,*) (BLOCKS(J,NBL1),J=1,3)
C
C          CHECK INPUT COORDINATES
C
      WRITE (*,104) NBLOCK,NBM,NBT
      WRITE (*,105) (BLOCKS(J,NBL1),J=1,3)
      DO 30 I = 1,NBLOCK
      N = NBL1 - I
      WRITE (*,106) N
      WRITE (*,107) BLOCKS(1,N),(TRANS(J,N),J=1,2),
     1              (BLOCKS(J,N),J=2,3)
   30 CONTINUE
C
      RETURN
C
C          TOO MANY BLOCKS REQUIRED FOR STORAGE ALLOCATED
C
   20 STOP
C
  107 FORMAT (14X,5F10.3)
  106 FORMAT (6X,I5)
  105 FORMAT (/14X,F10.3,20X,2F10.3)
  104 FORMAT (/5X,' CHECK INPUT DATA :'
     1        /9X,'NBLOCK = ',I3,'  NBM = ',I3,'  NBT = ',I3//
     2        8X,'BLOCK',6X,'XLEFT',5X,'XTRAN',4X,'XTRAN',5X,
     3        'XRIGHT',6X,'Y')
  103 FORMAT (/5X,' ENTER X1 AND X2 OF TRANSITION LEVEL: ')
  102 FORMAT (/5X,' ENTER WIDTH RATIO OF PIER AND MONOLITH: ')
  101 FORMAT (/5X,' ENTER NO. OF TRANSITION BLOCKS: ')
  100 FORMAT (/5X,' ENTER NO. OF BLOCKS OF MONOLITH: ')
   99 FORMAT (/' ENTER THE NUMBER OF BLOCKS IN THE DAM: ')
   97 FORMAT (/' ENTER THE DEFAULT UNIT WEIGHT: ')
   95 FORMAT (/5X,' ENTER X1,X2,Y, AND UNIT WEIGHT OF BLOCK NO.
     1        I2,': ')
   93 FORMAT (//5X,' ENTER X1,X2, AND Y AT THE CREST: ')
C
      END
C-----------------------------------------------------------------------
      SUBROUTINE BLCKVL (BLOCKS,TRANS,NBLOCK,NBM,NBT,ITYPE,WIDTHR,
     1                  UNITWT,WEIGHT)
C
```

```
C          COMPUTE THE LOCATIONS OF THE CENTROIDS AND
C          WEIGHTS OF THE BLOCKS
C
      DIMENSION BLOCKS(5,1),TRANS(2,1),UNITWT(1),WEIGHT(1)
C
C          LOOP OVER THE BLOCKS, ONE AT A TIME, TOP TO BOTTOM
C
      NBMT = NBM + NBT
      DO 10 J = 1,NBLOCK
          I = NBLOCK + 1 - J
          IF (I.LE.NBM .OR. I.GT.NBMT) GO TO 9
          CALL BCETRD (BLOCKS,TRANS,I,UNITWT(I),WIDTHR,
     1                 WEIGHT(I),BLOCKS(4,I),BLOCKS(5,I))
          GO TO 10
C
    9     TOP = BLOCKS(2,I+1) - BLOCKS(1,I+1)
          BOT = BLOCKS(2,I  ) - BLOCKS(1,I  )
          DX  = BLOCKS(1,I+1) - BLOCKS(1,I  )
          DY  = BLOCKS(3,I+1) - BLOCKS(3,I  )
C
          CALL CENTRD (TOP,BOT,0.0,DY,AREA,DUM,DUM,RY)
          BLOCKS(4,I) = BLOCKS(1,I) +
     1                  (2.0*DX*TOP + DX*BOT + TOP*BOT
     2                   + TOP*TOP + BOT*BOT)/
     3                  (3.0*(TOP + BOT))
          BLOCKS(5,I) = BLOCKS(3,I) + RY
C
          WEIGHT(I) = AREA*UNITWT(I)
          IF (I.GT.NBMT) WEIGHT(I) = WEIGHT(I)*WIDTHR
   10     CONTINUE
C
      RETURN
      END
C-----------------------------------------------------------------------
      SUBROUTINE FUNMOD (BLOCKS,NBLOCK,ITYPE,WEIGHT,WPHI,W1,W2,W3)
C
C          COMPUTE THE EFFECTIVE LATERAL LOAD FOR EACH BLOCK
C          AND THE TOTAL WEIGHT, EFFECTIVE EARTHQUAKE FORCE,
C          AND GENERALIZED WEIGHT OF THE DAM
C
      DIMENSION BLOCKS(5,1),WEIGHT(1),WPHI(1)
C
      HS = BLOCKS(3,NBLOCK+1) - BLOCKS(3,1)
      W1 = 0.0
      W2 = 0.0
      W3 = 0.0
C
C          LOOP OVER BLOCKS, ONE AT A TIME, BOTTOM TO TOP
C
      DO 10 I = 1,NBLOCK
          Y = (BLOCKS(5,I) - BLOCKS(3,1))/HS
          CALL PHIONE (Y,PHI,ITYPE)
          W  = WEIGHT(I)
          WP = W*PHI
          WPHI(I) = WP
```

```
          W1 = W1 + W
          W2 = W2 + WP
          W3 = W3 + WP*PHI
   10     CONTINUE
C
      RETURN
      END
C-----------------------------------------------------------------------
      SUBROUTINE PHIONE (Y,PHI,ITYPE)
C
C         OBTAIN THE ORDINATE OF THE FUNDAMENTAL VIBRATION MODE
C         OF THE DAM, USE THE STANDARD MODE SHAPE
C
      DIMENSION PHI1(22),PHI2(22)
      DATA DY/0.05/
      DATA PHI1/0.000 , 0.010 , 0.021 , 0.034 , 0.047 ,
     1            0.065 , 0.084 , 0.108 , 0.135 , 0.165 ,
     2            0.200 , 0.240 , 0.284 , 0.334 , 0.389 ,
     3            0.455 , 0.530 , 0.619 , 0.735 , 0.866 ,
     4            1.000 , 1.000 /
      DATA PHI2/0.000 , 0.016 , 0.030 , 0.048 , 0.070 ,
     1            0.094 , 0.123 , 0.155 , 0.192 , 0.232 ,
     2            0.277 , 0.327 , 0.381 , 0.440 , 0.504 ,
     3            0.572 , 0.646 , 0.725 , 0.816 . 0.909 ,
     4            1.000 , 1.000 /
C
      A   = Y/DY
      I   = IFIX(A) + 1
      A   = FLOAT(I) - A
      IF (ITYPE.EQ.1) GO TO 10
      PHI = A*PHI2(I) + (1.0-A)*PHI2(I+1)
      RETURN
   10 PHI = A*PHI1(I) + (1.0-A)*PHI1(I+1)
C
      RETURN
      END
C-----------------------------------------------------------------------
      SUBROUTINE DAMSTA (BLOCKS,NBLOCK,GAMMA,H,HB,PRESS,STRDUM)
C
C         COMPUTE THE STATIC STRESSES IN THE DAM DUE TO IMPOUNDED WATER
C
      DIMENSION BLOCKS(5,1),PRESS(1),STRDUM(2,1)
      EXTERNAL VALHST
C
C         COMPUTE STATIC STRESSES DUE TO IMPOUNDED WATER
C
      CALL CALHST (BLOCKS,NBLOCK,H,HB,GAMMA,PRESS)
      CALL STRPRS (BLOCKS,NBLOCK,PRESS,1,H,HB,VALHST,STRDUM)
C
C         COMPUTE STATIC STRESSES DUE TO TAILWATER -- NOT
C         IMPLEMENTED IN THIS VERSION OF THE PROGRAM
C
      RETURN
      END
C-----------------------------------------------------------------------
```

```
      SUBROUTINE DAMDYN (BLOCKS,NBLOCK,GAMMA,H,HB,PRESS,STRDUM)
C
C          READ HYDRODYNAMIC PRESSURE AND COMPUTE STRESSES
C          DUE TO THE HYDRODYNAMIC PRESSURE
C
      DIMENSION BLOCKS(5,1),PRESS(1),STRDUM(2,1)
      EXTERNAL VALHDY
C
      CALL REDHDY (BLOCKS,NBLOCK,H,HB,GAMMA,PRESS)
      CALL STRPRS (BLOCKS,NBLOCK,PRESS,1,H,HB,VALHDY,STRDUM)
C
      RETURN
      END
C-----------------------------------------------------------------------
      SUBROUTINE REDWAT (H,HB)
C
C          READ THE ELEVATIONS OF THE RESERVOIR
C
      WRITE (*,99)
      READ (*,*) H
      WRITE (*,98)
      READ (*,*) HB
C
     RETURN
C
   99 FORMAT (//' ENTER ELEVATION OF FREE-SURFACE: ')
   98 FORMAT (/ ' ENTER ELEVATION OF RESERVOIR BOTTOM:')
      END
C-----------------------------------------------------------------------
      SUBROUTINE COMSTA (NBLOCK,STR1,STR2,STR3,STRFAC,TITLE)
C
C          ADD STRESSES STR2 TO STR1 AND PUT IN STR3
C
      DIMENSION STR1(2,1),STR2(2,1),STR3(2,1)
      CHARACTER*40 TITLE
C
      WRITE (*,99) TITLE
C
      DO 10 J = 1,NBLOCK
          I = NBLOCK + 1 - J
          STR3(1,I) = (STR1(1,I) + STR2(1,I))*STRFAC
          STR3(2,I) = (STR1(2,I) + STR2(2,I))*STRFAC
          WRITE (*,98) I,STR3(1,I),STR3(2,I)
C
   10     CONTINUE
C
      RETURN
C
   99 FORMAT (//2X,A40//' BLOCK',5X,'UPSTREAM FACE',2X,'DOWNSTREAM FACE'
     1          /1X,40('-')/)
   98 FORMAT (3X,I2,10X,F8.3,9X,F8.3)
C
      END
C-----------------------------------------------------------------------

      SUBROUTINE CORPRS (BLOCKS,NBLOCK,H,HB,GAMMA,PRESS)
```

```
C
C        COMPUTE THE HYDRODYNAMIC PRESSURE ON THE UPSTREAM FACE OF A
C        RIGID DAM WITH INCOMPRESSIBLE WATER, USED FOR THE
C        COMPUTATION OF HIGHER MODE STRESSES
C
      DIMENSION BLOCKS(5,1),PRESS(1)
C
      DEPTH = H - HB
      IF (DEPTH.LE.0.0) RETURN
      NBL1 = NBLOCK + 1
      HS = BLOCKS(3,NBL1) - BLOCKS(3,1)
C
      DO 10 I = 1,NBL1
          PRESS(I) = 0.0
          Y = (BLOCKS(3,I) - HB)/DEPTH
          IF (Y.GT.1.0.OR.Y.LT.0.0) GO TO 10
          CALL POYFUN (Y,PO)
          PRESS(I) = GAMMA*DEPTH*PO
   10     CONTINUE
C
      RETURN
      END
C-----------------------------------------------------------------------
      SUBROUTINE POYFUN (Y,PO)
C
C        OBTAIN THE HYDRODYNAMIC PRESSURE ON A RIGID DAM WITH
C        INCOMPRESSIBLE WATER
C
      DIMENSION POY(22)
      DATA DY/0.05/,POY/0.742 , 0.741 , 0.737 , 0.731 , 0.722 , 0.711 ,
     1                  0.696 , 0.680 , 0.659 , 0.637 , 0.610 , 0.580 ,
     2                  0.546 , 0.509 , 0.465 , 0.418 , 0.362 , 0.301 ,
     3                  0.224 , 0.137 , 0.000 , 0.000 /
C
      A = Y/DY
      I = IFIX(A) + 1
      A = FLOAT(I) - A
      PO = A*POY(I) + (1.0-A)*POY(I+1)
C
      RETURN
      END
C-----------------------------------------------------------------------
      SUBROUTINE CALHST (BLOCKS,NBLOCK,H,HB,GAMMA,PRESS)
C
C        COMPUTE THE HYDROSTATIC PRESSURE ON THE FACE OF THE DAM
C
      DIMENSION BLOCKS(5,1),PRESS(1)
C
C        LOOP OVER THE BLOCK LEVELS, ONE AT A TIME, BOTTOM TO TOP
C
      NBL1 = NBLOCK + 1
      DO 10 I = 1,NBL1
          PRESS(I) = 0.0
          Y = BLOCKS(3,I)
          IF (Y.LT.H.AND.Y.GE.HB) PRESS(I) = GAMMA*(H-Y)
```

```
   10      CONTINUE
C
      RETURN
      END
C-----------------------------------------------------------------------
      SUBROUTINE REDHDY (BLOCKS,NBLOCK,H,HB,GAMMA,PRESS)
C
C          READ AND COMPUTE THE HYDRODYNAMIC PRESSURE AT THE
C          BLOCK LEVELS ON THE UPSTREAM FACE OF THE DAM
C
      DIMENSION BLOCKS(5,1),PRESS(1)
C
      DEPTH = H - HB
      IF (DEPTH.EQ.0.0) RETURN
C
      NBL1 = NBLOCK + 1
      HHS  = DEPTH/(BLOCKS(3,NBL1) - BLOCKS(3,1))
      HHS2 = HHS*HHS
C
C          LOOP OVER BLOCK LEVELS, ONE AT A TIME, TOP TO BOTTOM
C
      WRITE (*,99)
      DO 10 J = 1,NBL1
          I = NBLOCK + 2 - J
          PRESS(I) = 0.0
          Y = (BLOCKS(3,I) - HB)/DEPTH
          IF (Y.GT.1.0.OR.Y.LT.0.0) GO TO 10
C
C              READ PRESSURE COEFFICIENT AND COMPUTE HYDRODYNAMIC
C              PRESSURE AT THE BLOCK LEVEL
C
          WRITE (*,98) Y
          READ (*,*) P
          PRESS(I) = GAMMA*DEPTH*HHS2*P
C
   10     CONTINUE
C
      RETURN
C
   99 FORMAT (//' ENTER THE HYDRODYNAMIC PRESSURE FOR THE'/
     1            ' FUNDAMENTAL VIBRATION MODE OF THE DAM')
   98 FORMAT (/5X,' ENTER THE PRESSURE ORDINATE FOR Y/H =',
     1           F5.3,': ')
C
      END
C-----------------------------------------------------------------------
      SUBROUTINE STRLOD (BLOCKS,NBLOCK,LOADS,VALUES,STRESS)
C
C          COMPUTE THE NORMAL STRESSES DUE TO LOADS APPLIED AT THE
C          CENTROID OF THE BLOCKS
C
      DIMENSION BLOCKS(5,1),LOADS(1),STRESS(2,1)
      REAL LOADS,M
C
      HSUM  = 0.0
```

```
      HYSUM = 0.0
      VSUM  = 0.0
      VXSUM = 0.0
C
C          LOOP OVER BLOCKS,ONE AT A TIME, TOP TO BOTTOM
C
      DO 10 J = 1,NBLOCK
          I = NBLOCK + 1 - J
C
C              OBTAIN THE LOADS AT THE CENTROID OF BLOCK I
C
          CALL VALUES (I,LOADS,V,H)
          HSUM  = HSUM  + H
          HYSUM = HYSUM + H*BLOCKS(5,I)
          VSUM  = VSUM  + V
          VXSUM = VXSUM + V*BLOCKS(4,I)
C
C              COMPUTE THE BENDING MOMENT AND STRESSES AT THE
C              BOTTOM OF BLOCK I
C
          M = HYSUM - VXSUM - BLOCKS(3,I)*HSUM
     1        + 0.5*(BLOCKS(2,I)+BLOCKS(1,I))*VSUM
C
          T = BLOCKS(2,I) - BLOCKS(1,I)
          M = 6.0*M/(T*T)
          STRESS(1,I) = VSUM/T + M
          STRESS(2,I) = VSUM/T - M
C
   10     CONTINUE
C
      RETURN
      END
C-----------------------------------------------------------------------
      SUBROUTINE STRPRS (BLOCKS,NBLOCK,PRESS,IUPDN,H,HB,VALUES,STRESS)
C
C          COMPUTE THE NORMAL STRESSES DUE TO PRESSURE APPLIED
C          AT THE FACE, UPSTREAM (IUPDN=1) OR DOWNSTREAM (IUPDN=2)
C          OF THE BLOCKS
C
      DIMENSION BLOCKS(5,1),PRESS(1),STRESS(2,1)
      REAL M
      LOGICAL YCOMP
C
      HSUM  = 0.0
      HYSUM = 0.0
      VSUM  = 0.0
      VXSUM = 0.0
C
      YB = BLOCKS(3,NBLOCK+1)
C
C          LOOP OVER BLOCKS, ONE AT A TIME, TOP TO BOTTOM
C
      DO 40 J = 1,NBLOCK
C
          I = NBLOCK + 1 - J
```

```
      YBT = YB
      YB  = BLOCKS(3,I)
C
      IF (YB.GE.H.OR.YBT.LE.HB) GO TO 30
C
C         THE BLOCK TOUCHES WATER, OBTAIN THE WATER PRESSURE
C         AT THE TOP AND BOTTOM OF THE BLOCK
C
      CALL VALUES (I,PRESS,P1,P2,YCOMP)
      DX = 0.0
      IF (YCOMP) DX = BLOCKS(IUPDN,I+1) - BLOCKS(IUPDN,I)
      DY = BLOCKS(3,I+1) - BLOCKS(3,I)
      IF (YBT.LE.H) GO TO 10
C
C         TOP OF WATER IS IN BLOCK, MODIFY TOP PRESSURE POINT
C
      DUM = H - YB
      DX  = DX*DUM/DY
      DY  = DUM
      P1  = 0.0
      GO TO 20
C
C         CHECK THAT BOTTOM OF WATER CORRESPONDS TO A BLOCK
C
   10 IF (YB.LT.HB) WRITE (*,99)
C
C         COMPUTE PRESSURE AND FORCES ACTING ON BLOCK I
C
   20 CALL CENTRD (P1,P2,DX,DY,H1,V,RX,RY)
C
C         COMPUTE THE STRESS RESULTANTS AT THE BOTTOM OF BLOCK
C
      HSUM  = HSUM  + H1
      HYSUM = HYSUM + H1*(YB+RY)
      VSUM  = VSUM  + V
      VXSUM = VXSUM + V*(BLOCKS(IUPDN,I)+RX)
C
C         COMPUTE THE BENDING MOMENTS AND STRESSES AT THE
C         BOTTOM OF BLOCK I
C
   30 M = HYSUM - VXSUM - BLOCKS(3,I)*HSUM
     1       + 0.5*(BLOCKS(2,I)+BLOCKS(1,I))*VSUM
C
      T = BLOCKS(2,I) - BLOCKS(1,I)
      M = 6.0*M/(T*T)
      STRESS(1,I) = VSUM/T + M
      STRESS(2,I) = VSUM/T - M
C
   40 CONTINUE
C
    RETURN
C
   99 FORMAT (//' ERROR IN MODEL - RESERVOIR BOTTOM DOES NOT'/
     1          ' COINCIDE WITH THE BOTTOM OF A BLOCK'/)
C
```

```
      END
C-----------------------------------------------------------------------
      SUBROUTINE VALWGT (I,LOADS,V,H)
C
C         OBTAIN THE WEIGHT OF BLOCK I
C
      DIMENSION LOADS(1)
      REAL LOADS
C
      H = 0.0
      V = - LOADS(I)
C
      RETURN
      END
C-----------------------------------------------------------------------
      SUBROUTINE VALHOR (I,LOADS,V,H)
C
C         OBTAIN THE EFFECTIVE LATERAL FORCE ON BLOCK I
C
      DIMENSION LOADS(1)
      REAL LOADS
C
      H = LOADS(I)
      V = 0.0
C
      RETURN
      END
C-----------------------------------------------------------------------
      SUBROUTINE VALHST (I,PRESS,P1,P2,YCOMP)
C
C         OBTAIN THE HYDROSTATIC PRESSURE ON BLOCK I
C
      DIMENSION PRESS(1)
      LOGICAL YCOMP
C
      P1 = PRESS(I+1)
      P2 = PRESS(I  )
      YCOMP = .TRUE.
C
      RETURN
      END
C-----------------------------------------------------------------------
      SUBROUTINE VALHDY (I,PRESS,P1,P2,YCOMP)
C
C         OBTAIN THE HYDRODYNAMIC PRESSURE ON BLOCK I
C
      DIMENSION PRESS(1)
      LOGICAL YCOMP
C
      P1 = PRESS(I+1)
      P2 = PRESS(I  )
      YCOMP  = .FALSE.
C
      RETURN
      END
```

```
C-----------------------------------------------------------------------
      SUBROUTINE CENTRD (P1,P2,DX,DY,PX,PY,RX,RY)
C
C          COMPUTE THE RESULTANT PRESSURE FORCE ON A SURFACE.
C          ALSO LOCATES THE VERTICAL CENTROID OF A BLOCK
C
      A  = 0.5*(P1+P2)
      PX =    A*DY
      PY = - A*DX
C
      RX = 0.0
      RY = 0.0
      IF (A.EQ.0.0) RETURN
C
      A  = (2.0*P1+P2)/(6.0*A)
      RX = A*DX
      RY = A*DY
C
      RETURN
      END
C-----------------------------------------------------------------------
      SUBROUTINE BCETRD (BLOCKS,TRANS,JB,UW,WRATIO,WT,CX,CY)
C
C          COMPUTE THE WEIGHT AND LOCATE THE WEIGHT CENTROID
C          OF A TRANSITION BLOCK
C
      DIMENSION BLOCKS(5,1),TRANS(2,1)
      UWR = UW*WRATIO
      CALL WCETRD (BLOCKS(1,JB),TRANS(1,JB),BLOCKS(1,JB+1),TRANS(1,JB+1)
     1             ,BLOCKS(3,JB),BLOCKS(3,JB+1),UWR,WT1,C1X,C1Y)
      CALL WCETRD (TRANS(1,JB),TRANS(2,JB),TRANS(1,JB+1),TRANS(2,JB+1)
     1             ,BLOCKS(3,JB),BLOCKS(3,JB+1),UW ,WT2,C2X,C2Y)
      CALL WCETRD (TRANS(2,JB),BLOCKS(2,JB),TRANS(2,JB+1),BLOCKS(2.JB+1)
     1             ,BLOCKS(3,JB),BLOCKS(3,JB+1),UWR,WT3,C3X,C3Y)
      WT  = WT1+WT2+WT3
      CX  = (C1X*WT1+C2X*WT2+C3X*WT3)/WT  - BLOCKS(1,1)
      CY  = (C1Y*WT1+C2Y*WT2+C3Y*WT3)/WT  - BLOCKS(3,1)
      RETURN
      END
C-----------------------------------------------------------------------

      SUBROUTINE WCETRD (X1,X2,X3,X4,Y1,Y2,UW,WT,CX,CY)
C
C          COMPUTE THE WEIGHT AND THE WEIGHT CENTROID OF
C          A BLOCK OF TRIANGULAR OF TRAPEZOIDAL SHAPE
C
      DX12  = X2-X1
      DX34  = X4-X3
      DY    = Y2-Y1
      C1X   = (X1+X2+X4)/3.
      C1Y   = (Y1+Y1+Y2)/3.
      C2X   = (X1+X3+X4)/3.
      C2Y   = (Y1+Y2+Y2)/3.
      AREA1 = 0.5*DX12*DY
      AREA2 = 0.5*DX34*DY
      WA1   = UW*AREA1
```

```
      WA2    = UW*AREA2
      WT     = WA1+WA2
      IF (WT.NE.0.0) GO TO 10
      CX     = 0.5*(X1+X3)
      CY     = 0.5*(Y1+Y2)
      RETURN
   10 CX     = (C1X*WA1+C2X*WA2)/WT
      CY     = (C1Y*WA1+C2Y*WA2)/WT
      RETURN
      END
```

APPENDIX F –RELATING POTENTIAL FAILURE MODES AND EVENT TREE SEQUENCES TO APPROPRIATE STRUCTURAL ANALYSES

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RELATING POTENTIAL FAILURE MODES AND EVENT TREE SEQUENCES TO APPROPRIATE STRUCTURAL ANALYSES

This Appendix will relate concepts discussed in this manual and the relationship with the Potential Failure Mode Analysis (PFMA) and Risk Analyses concepts presented in CWC-Risk 2018 and FEMA 2015. The objective is to guide structural analyses to select appropriate structural analysis methods to address key events along potential failure mode paths. The purpose of this Appendix as stated in FEMA 2015 is to stress the importance of understanding the sequences of events leading to failure of concrete dams and selecting analysis methods that address these specific events. The selected analysis method may range from straightforward to complex depending on the event of the potential failure mode being analyzed. It is stressed that a less complex analysis with less uncertainty is the preferred strategy. Each dam is unique and has its own issues; therefore, it is important for the engineer to understand the potential failure modes and sequences of events that enable them.

It is further stated in FEMA 2015 that failure is considered the uncontrolled release of the reservoir. However, this definition of failure may or may not always be the case given the purpose or hazard of a structure or given the regulatory requirements for a structure. How do we determine if a concrete dam can fail? Failure results from sequences of events that must follow one upon another. Because a dam cannot fail without the full chain of events, conclusively ruling out any event justifies concluding that the dam will not fail. This Appendix introduces event trees that are pictorial representations of the sequences of events (called nodes) leading to failure. The possibility of each node occurring is evaluated by analyses. Since finding that any particular node is impossible to occur rules out failure, often; therefore, not all nodes need to be analyzed. As will be shown, some analysis techniques can address some of the nodes in an event tree, and some cannot.

As described in CWC-RISK 2018, a PFMA is the process of identifying and fully describing potential failure modes. A failure mode is a unique set of conditions and/or sequence of events that could result in failure (or breach) of the dam, where failure is defined as an uncontrolled, potentially life-threatening release of the reservoir for flood risk management projects.

Identifying, fully describing, and evaluating site-specific potential failure modes and sequences leading to failure are arguably the most important initial steps in conducting a structural analysis for a concrete dam and deciding the appropriate data needs, field investigations, laboratory testing, structural analyses, and ultimately monitoring. This first step should be done in a diligent and thorough manner that produces valuable information for subsequent safety or risk assessment. As a result, the assessment will result in more accurate, substantiated, and dependable conclusions for dam safety decisions. The process this Appendix suggests is to: (1) define failure criteria, (2) identify potential failure modes, (3) develop a sequence of events (nodes) for a failure to transpire, (4) select a node with the most likely chance to circumvent the failure process, (5) select a structural analyses method to compute the response of the dam at that node, (6) quantify the uncertainty, and (7) build the case that the failure process terminates or does not terminate at that node or builds the case of a given probability of failure at that node. These steps are discussed further in more detail in CWC-RISK 2018. This Appendix will further supplement steps 3, 4, and 5.

Initiating Events

Failures start with some initiating event that causes an adverse change in the structure. Typically these include loads during normal operating conditions, flood events, or seismic loadings.

Normal operating conditions are events that can occur during the usual day to day operation of the facility. Something must change or be "different than expected" for a potential failure mode to occur during normal operating conditions. Generally material properties of the concrete or foundation (density, compressive strength, tensile strength, modulus of elasticity, or shear strength) do not change over time; except, if there is a secondary action happening, such as alkali-aggregate reaction (AAR), freeze-thaw (FT), leaching or piping of material, calcification, erosion, corrosion, etc. AAR causes expansive forces, FT causes reduction in concrete strength and geometric thickness, piping reduces the effective material volume, calcification reduces in drain efficiency and increases uplift pressures, and corrosion degrades steel and reduces strength.

Actions that might be "different than expected" are 1) actual material properties are different than assumed in design, 2) unexpected loads overstress the materials, 3) faulty instrumentation gives incorrect readings, 4) power outages, 5) equipment malfunction, 6) circumstances inhibit access to the site or controls, 7) drain clogging leads to increased uplift pressures, 8) degradation of the grout curtain leading to increased seepage and increased uplift pressures, 9) alkali-aggregate reaction in the concrete leads to reduced concrete strength, 10) corrosion of the reinforcing steel leads to reduced capacity of the member, or 11) debris blocks the spillway reducing flow. Some of these changing conditions can be identified and quantified with diligent, long-term instrumentation, monitoring, and inspections. Some of the conditions that are "different than expected" can result in being similar to other initiating events. For instance, if reservoir water cannot be released through the spillway during normal operating conditions due to debris blockage, the rise in reservoir level could be the same as a higher return period flood. The failure probability of the dam for these 2 conditions (reservoir rise due to debris blockage vs. flood conditions) is the same because the reservoir is at the same level. However, the risks could be different because of the different probability for occurrence for debris blockage or flooding. Also, the loss of life consequences could be different for the 2 scenarios because of different warning times.

Hydrologic or flood events occur when the reservoir rises above the normal operating condition. This causes increased hydrostatic loads on the structure, possibly overtops the dam and erodes the foundation, or possibly increases flow through the spillway inducing scour.

Seismic events occur during an earthquake. This causes increased inertial forces on the structure that might lead to overstressing or induces displacement of the foundation under the dam inducing misalignment. Typically earthquakes are considered to occur during normal operating conditions and not flood conditions. Failures can occur during an earthquake or after an earthquake from static loads or aftershocks if the structure was damaged enough during the earthquake. Historically concrete gravity dams and spillways have performed very well during earthquakes. Generally the duration of strong shaking is not long enough to cause failures.

Initiating events caused by operation and maintenance (O&M) issues or other reasons may or may not directly cause a failure or are not in themselves failures, but they may start a potential failure mode. For instance, a bad or failed instrument may not provide the correct uplift under a dam, but if the dam is stable under full uplift conditions, the bad instrument reading does not matter. Or, scour of the spillway apron may not induce an uncontrolled release of the reservoir if the ogee spillway section remains stable. Likewise, the inability to operate spillway gates due to a power failure might lead to higher reservoir levels than expected, but may not induce an uncontrolled release of the reservoir if the dam remains stable.

Sequence of Events (Event Trees)

The more critical potential failure modes identified in a PFM workshop would then be described in detail. The entire sequence of events (nodes) leading to failure is developed: (1) to ensure that there is a common understanding of the failure mode, (2) to ensure reviewers in the future will have a clear understanding of what was being considered, and (3) because an event tree can be developed to further explain and show the sequences.

For instance, during a flood, a dam might fail by sliding. A sequence of events (nodes) for this potential failure mode that would need to occur might be: (node 1) flooding with reservoir above a given elevation yet below the top of the parapet walls, (node 2) the increased load causes tension at the heel of the dam causing the concrete to crack or the interface to open; (node 3) uplift pressures increase in the crack, causing the crack to propagate through the thickness of the dam; (node 4) the driving forces (and reduced normal force on the crack) on the dam are higher than the shear strength along the cracked surface, and sliding starts; (node 5) there are no mechanisms that stop the sliding, and the dam fails.

More importantly, developing the entire sequence of events provides a clearer understanding for the structural engineer of what is needed from a structural analysis perspective. In the above example, a structural analysis targeted on node 2 would compute the tensile stress at the heel due to flood loading above the given elevation, on node 3 would compute crack length and uplift pressures, on node 4 would compute sliding factors of safety, or on node 5 would identify mechanisms that might stop sliding if sliding starts such as 3 dimensional effects in a narrow canyon or side friction with adjacent monoliths.

The following are more examples of initiating events and the sequences that need to occur for failure:

- Deterioration of concrete under all loads.—This potential failure mode results from deteriorating and weakening of the concrete at specific locations. The deterioration of the concrete is allowed to progress to the point where the applied loads are greater than the strength of the concrete, and failure occurs. The loads might be during normal operating conditions, during a flood, or during a seismic event. The critical initiating event occurs when the strength of the concrete is less than the applied stress. The structure might fail in different ways depending on whether the structure is loaded in tension or compression, or by misalignment.

1. Concrete deteriorates and loses strength over time at a given location
2. An earthquake occurs on a given fault producing a level of acceleration
3. Structure significantly deforms (more than usual)
4. Misalignment causes redistribution of load
5. Structure cannot carry redistributed load
6. Structure fails

- Flood loading overloads concrete dam.—This potential failure mode occurs during a flood when increased water and uplift overload the dam causing cracking of concrete to the point that the dam fails by sliding.

1. Reservoir water surface at or above a given level
2. Tensile stresses increase on upstream face of the dam
3. Concrete cracks
4. Uplift increases in the crack
5. Dam cracks through the thickness of the dam
6. Sliding commences and structure cannot redistribute load, and dam fails

- Uplift increases under concrete dam.—This potential failure mode occurs when the drains in the dam become ineffective due to plugging and uplift pressures increase under the dam. The normal force along a horizontal slide plane reduces the frictional resistance, and the dam fails due to sliding.

1. Drains become less effective causing an increase in uplift pressures
2. Reduced normal force on horizontal slide plane decreases frictional resistance
3. Sliding commences, and dam fails during normal operating conditions

- Earthquake overloads foundation blocks and causes failure of dam.—This potential failure mode is caused by an earthquake that increases loads from the dam into the foundation and also causes inertia forces of the foundation blocks. It has been established that there are removable foundation blocks in the abutment.

1. An earthquake increase forces from the dam into the foundation blocks
2. Foundation blocks become unstable with 40-degree friction and move
3. Deformations occur in the dam with 4 feet of foundation movement
4. Dam fails due to load redistribution

An event tree is a graphical representation, available to the engineer, that provides an efficient way to organize the chronological sequence of events for a particular potential failure mode from the initiating cause on the left, through a series of linked events (nodes or branches), to the failure or no failure condition on the right (see Figure F-1). Each node represents an event or condition with possible outcomes or states that need to exist for failure to ultimately occur or not occur. Event trees are easily understood because they portray the chronological sequence of events that must occur for failure to happen. Event trees aid in the decomposition of failure modes. This decomposition aids in the structural analyst's understanding the failure mode and also in briefings to management.

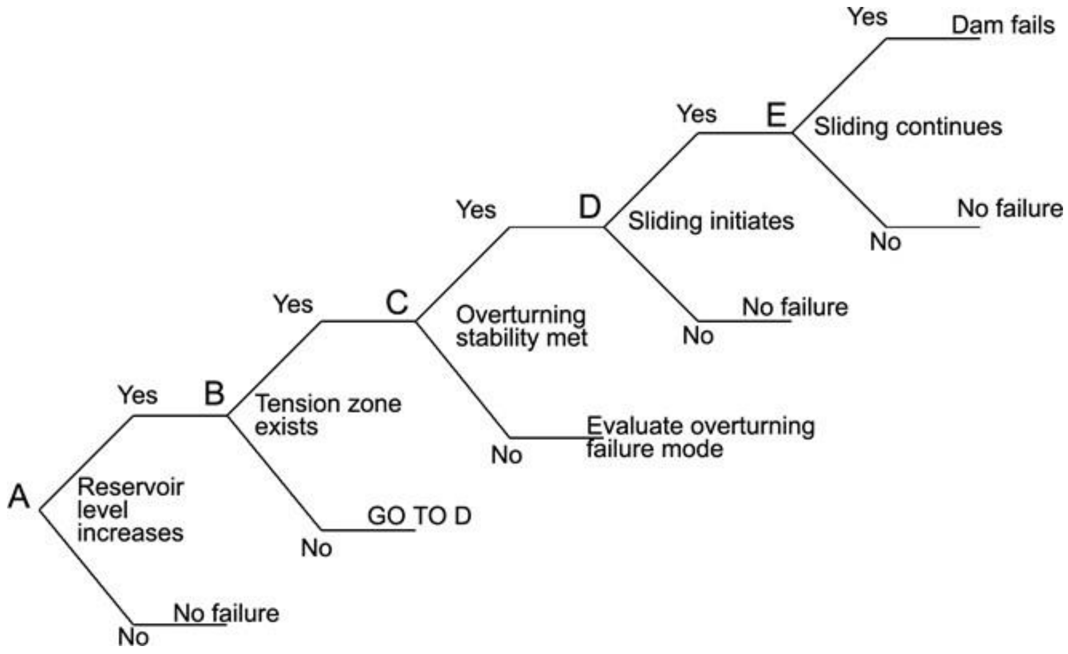

Figure F-1 - An example of an Event Tree for sliding and overturning of a gravity dam

An event tree should be developed for each potential failure mode.

Relating Structural Analyses to Failure Modes

Potential failure modes characterize how a structure might fail and identify the sequence of events (nodes) leading to failure. Structural analysis is used to determine the structural response of a given node. For instance, the first node on an event tree might be that an increased load causes the concrete to crack. There are various methods to determine if the concrete will crack such as structural analysis, physical testing, or scale-model testing. Structural analysis uses either limit equilibrium analysis or various types of finite element methods (FEM); namely, 2-dimensional (2D) linear elastic, 2D nonlinear, 3D linear-elastic, or 3D nonlinear. It must be realized that each one of these analysis methods has a certain level of applicability and represents a certain level of reality. The technique chosen should be the one that answers the specific question being asked. These methods also vary greatly in cost, difficulty, and level of effort. The acceptability of the results depends on the acceptability of the material properties being used, the acceptability of the loading condition, and the way the structure was modeled. An assumed, yet conservative, material property may be sufficient without testing to satisfy stability requirements.

Traditional limit equilibrium as described in this manual may be appropriate to provide the structural response at many nodes along an event tree. For instance for a gravity dam, limit equilibrium provides the reservoir level that causes tension at the heel, the distance along the base of the dam might crack, or the sliding factors of safety. However, limit equilibrium analyses assumes the base of the dam (or slide plane) is planar, the normal stress distribution along the dam base is trapezoidal, tension between the dam and foundation is not allowed, and 2D is appropriate. Also, limit equilibrium analyses are not appropriate for seismic analyses.

Linear analyses assume the dam and foundation are homogeneous and the material act elastically. Both these assumptions are not valid for concrete dams and foundations; however, linear analyses have their place in structural analysis toolboxes. Results from linear finite element analyses provide an essential baseline to compare to nonlinear analyses, provide natural frequency of the structure, and provide stresses and displacements in the dam. If results stay in the linear range, it may not be necessary to do nonlinear analyses. Nonlinear analyses that are run in "linear mode" should produce results comparable to linear analysis results. Stress levels computed that are unrealistic or well into the nonlinear range of the material justify performing nonlinear analyses. However, linear finite element analysis does not address failure. It answers a different question, providing stresses and deflections subject to a different set of assumptions. The assumption underlying linear finite element analysis is that the dam and foundation form a continuous, linear, elastic solid. Cracking, crushing, or slipping cannot be directly modeled. Because it is not sufficient in itself to evaluate a dam at impending failure, it cannot be compared directly with limit equilibrium analysis. Tension (sometimes unrealistic tension) is computed between the dam and foundation or within the dam. However, even with these limitations, linear finite element analysis can provide some insight into dam behavior and also model the 3D effects of a dam. For dynamic loading, the vibration mode shapes and natural frequencies generated by linear analysis are of some interest. For reinforced concrete structures such as slab and buttress dams, moment and force resultants from linear models are useful in evaluating the adequacy of members.

Nonlinear analyses can provide the 3-dimensional static and dynamic structural response of a concrete dam give its nonlinear features: nonlinear materials (concrete and rock) and discontinuities (cracks, vertical contraction joints, unbonded lift joints, foundation discontinuities). Nonlinear analyses can also include the dimension of time (duration) of an earthquake of flood and predict the amount of sliding. Nonlinear analyses have their own set of difficulties of characterizing the material properties, applying the loads, running appropriate solution time steps, and accurately predicting nonlinear behavior. However, nonlinear analyses do provide insights into the possible responses of a dam.

Material Properties

Evaluation of dam structures requires that the engineer make assumptions and utilize various parameters and analysis methods to develop a reasonably accurate representation of structural behavior. The impacts of particular parameters on analysis results vary based on the type of analysis being performed. Some typical structural parameters are:

- Joint friction angle and cohesion (parent concrete, lift joints, and rock discontinuities)
- Compressive strength (concrete and rock)
- Tensile strength (parent concrete, lift joints)
- Modulus of elasticity (concrete and rock)
- Poisson's ratio (concrete and rock)
- Coefficient of thermal expansion (concrete)
- Unit weight (concrete, rock, and water)
- Damping (system)
- Geometry (dam and foundation)
- Mesh coarseness (dam, foundation, and water)
- Boundary conditions (foundation and reservoir)

Accurate determination of parameters is very often a daunting task. For example, to determine a shear strength to be used in a sliding analysis, a coring program through the slide plane must be undertaken. A sample must be recovered intact and relatively undisturbed. If asperities, roughness on a joint plane, are to be considered, the sample must be large enough to capture the asperities. One sample can be tested multiple times, but it will only yield peak shear strength on its initial shearing. Sufficient samples must be obtained to account for the normal variation. Because of the difficulty in recovering and testing statistically significant samples, it behooves the analyst to understand what effect parameter variation would have on analysis results. Some parameters have little effect on analysis outcome and therefore do not need to be determined with great accuracy. Some parameters are important in a certain range but not in others.

Structural Analyses Cases and Confidence

A structural analyst can help the potential failure mode process by performing some sensitivity studies before the PFMA workshop. For instance, various limit equilibrium analyses or linear finite element studies can answer the questions:

- At what reservoir level does the heel go into tension?
- What is the dam stability with functioning drains and plugged drains?
- How sensitive is the dam to sliding for various levels of shear strength?
- What magnitude of tensile stress develops in the dam at various earthquake levels?

The goal of a structural analyst is to perform an analysis appropriate to the level of study and focused on capturing the desired structural response at an event node associated with a potential failure mode, and to portray the uncertainties and assumptions to decisionmakers. Choosing the appropriate analysis method is key in answering a desired question about the dam. The problem being solved should dictate the analysis method selected. Once the analysis method is chosen, the accuracy or usefulness of the solution depends on the understanding of the material properties, boundary conditions, and loads input into the program. The many choices in analyses cause variability in the computed results. These might include, but are definitely not limited to, choices between:

- Average, generic material properties instead of site-specific tested material properties
- Using a massless foundation instead of a foundation with mass
- Viscous damping or Rayleigh damping
- Westergaard's added mass or compressible fluid elements for hydrodynamic interaction
- Modeling the dam as a homogeneous structure instead of modeling geometric nonlinearities in the dam such as contraction joints or unbonded lift joints
- Linear or nonlinear material properties
- A response spectrum analysis or a time-history analysis
- Modal superposition or direct time integration
- A coarse or a fine finite element mesh
- Various types of solid elements
- Ground motions generated at 0.02-second or at 0.005-second intervals
- Modeling 3D or only 2D canyon effects
- Applying ground motions in three orthogonal directions or in only one direction
- Using appropriate ground motions for the site including spatial variations.

The analyst may have to perform multiple structural analyses varying individual parameters to obtain the sensitivity of the parameter on the results.

Example of Earthquake Induced Sliding in the Foundation

This potential failure mode is due to a seismic event with sufficient energy to displace a removable rock block of sufficient size in an abutment causing loss of foundation support for a curved gravity dam (see Figure A2). With loss of abutment support, sections of the dam may not be stable. The dam and foundation blocks may slide together and fail as a unit.

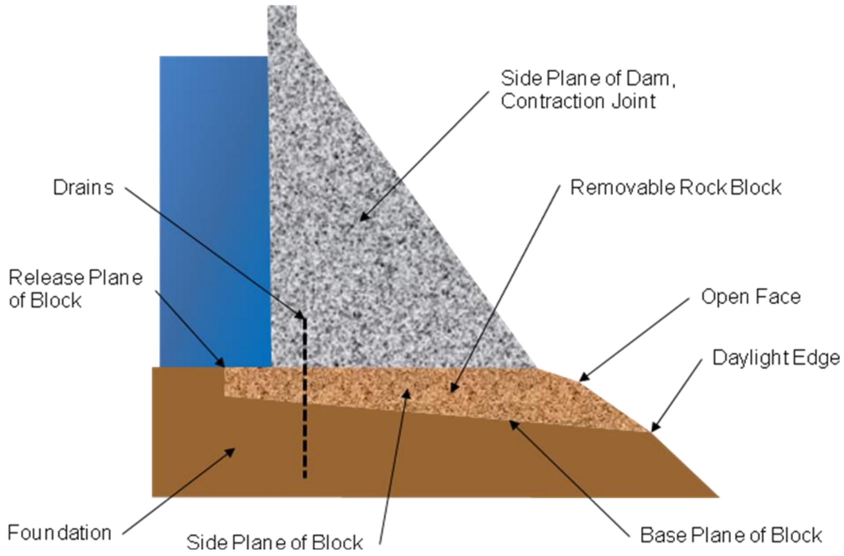

Figure F-2.—Schematic of a dam built on a removable rock block in the foundation of sufficient size to affect dam stability if the block moves.

The key point in the description of this potential failure mode is that the rock block is potentially removable and is of sufficient size to affect the stability of the dam. For this example, it is assumed that no potential failure mode is associated with rock blocks if there is no removable block in the foundation. A removable block has a base plane, side plane(s), and a release plane, as well as a free surface for the block to slide toward. The base plane of the block is a relatively horizontal surface forming the bottom of the block. The side planes of the block are steeply sloping, diverging surfaces oriented in the stream direction. The release plane is a steeply sloping surface oriented in the cross-canyon direction and positioned at the upstream extent of the block. Movement of the rock block would slide on both or either of the base and side planes and would pull away from the release plane. An open face is an unrestrained face. There is also no potential failure mode if the block is too small to affect the stability of the dam. In this case, the dam can bridge across the block. Considerable field work may be necessary to determine the likelihood that a block exists and that this potential failure mode is possible in the foundation. Movement of the rock block is a function of the driving forces on the block and the resisting forces as described below. Figure F-3 shows graphically what might happen if the rock block slides, the loading path changes, the shearing resistance changes, the concrete fractures, and the drainage system undergoes disruption. In addition, dilation of the foundation planes may result in increased seepage and a change in water pressures including the development of full hydrostatic pressure on the release plane.

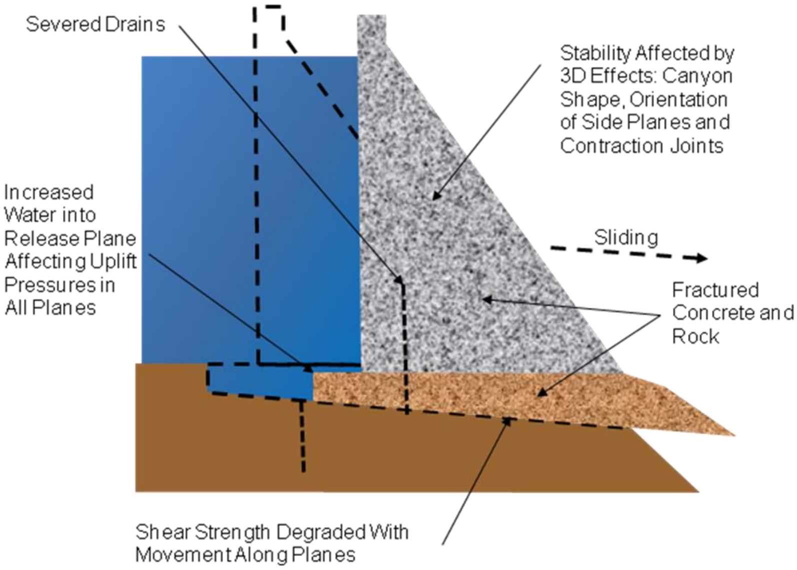

Figure F-3—Potential failure mode of sliding of a removable block in the foundation induced by a seismic event and postseismic considerations.

The event tree for this potential failure mode is shown in Figure F-4 and has five nodes along the tree. In this case, it has been determined that there is a removable block in the foundation and the potential failure mode exists. The analyst could start at any point along the event tree to attempt to disprove failure.

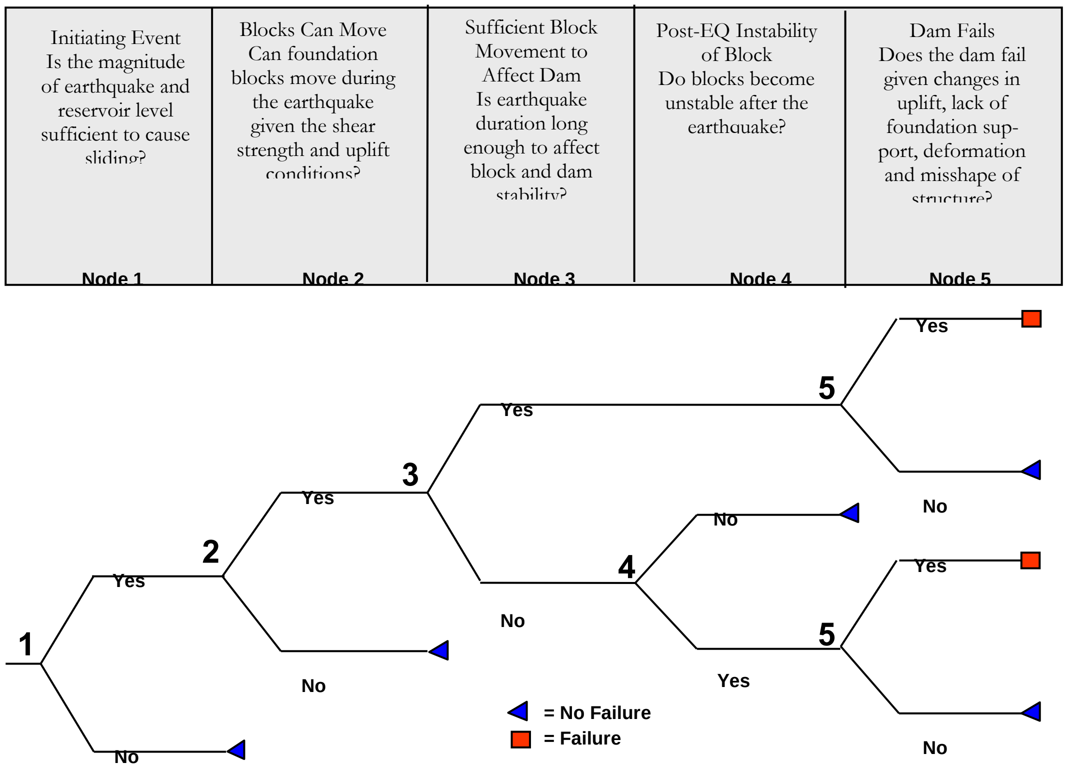

Figure F-4—Event tree for this potential failure mode.

Node 1—Initiating Event—Earthquake Load with Normal Operating Loads

Node 1 of the event tree considers if the earthquake has sufficient magnitude along with the corresponding static loads to potentially cause movement of the foundation block. When appropriate, static operating loads include gravity, reservoir, ice, tailwater, silt, temperature, uplift, and other loads during normal operations. The dynamic loads include inertial forces of the dam onto the foundation block, inertial forces of the foundation, and the hydrodynamic interaction of the reservoir. The seismologist would determine the magnitude of potential earthquakes at the site. The structural analyst would determine the possible damage to the structure given the levels of earthquakes being postulated and the material properties.

Node 2—Movement of Foundation Block Commences

Node 2 of the event tree considers if the driving forces on the foundation blocks are sufficient to overcome the sliding resistance and sliding initiates. Movement of foundation blocks is a function of the magnitude and orientation of the applied loads, the cyclical nature and frequency content of the ground motion, the shear strength along sliding planes, and the orientation of the sliding planes. Shear strength along sliding planes is a function of joint roughness and infilling material.

- The geologist and geotechnical engineer would determine the shear strength along the foundation discontinuities.
- The structural analyst could perform limit equilibrium or linear finite element to determine the sliding factors of safety for various levels of earthquakes and different friction angles. This will answer the question, "Can an earthquake cause sliding along the foundation discontinuity?".

Node 3—Sufficient Block Movement Affects Dam

Node 3 of the event tree determines if the earthquake lasts long enough to cause sufficient sliding of the foundation blocks to affect the stability of the dam. The earthquake must have sufficient duration to induce sufficient movements in the foundation to have the potential to cause a dam failure. One large pulse of the earthquake may cause initial foundation block movement, but if the movement is too small and not sustained, redistributed loads within the dam may prevent failure. Longer duration earthquakes can cause larger movements in the foundation such that loads cannot be redistributed in the dam and thus might cause failure during the earthquake. For this reason, the event tree splits into two possible scenarios at node 3. If the earthquake has sufficient duration to move the foundation block enough to affect the stability of the dam, the tree progresses to node 5, where the stability of the dam is evaluated during the earthquake. If the earthquake does not have enough duration to fail the dam during the earthquake, the event tree progresses to node 4, where the postearthquake stability of the foundation block is evaluated due to possible changes in static load and sliding resistance.

- The structural engineer must decide the appropriate analysis approach. A linear finite element analysis will not answer the question about how far the structure will slide. Forces from a linear analysis coupled with a Newmark sliding study can determine how far the block might slide, but may not be too accurate. A 2-dimensional nonlinear finite element study with contact surfaces along the slide plane would compute the amount of sliding but not take into account any 3-dimensional effects. A 3-dimensional nonlinear finite element study would take into account all the 3-dimensional effects.

- If the dam moves too far, a field program might be initiated to determine the shear strength properties along the slide plane.

Node 4—Foundation Block Slides After the Earthquake (Postearthquake)

Node 4 of the event tree deals with postseismic stability if the foundation blocks do not slide enough during the earthquake to adversely affect dam stability. Movements of the foundation block may change stability conditions, increasing uplift and waterflow around the block, crushing or fracturing rock blocks, or reducing shear strength from sliding. Postseismic stability considerations might include aftershocks. Uplift might increase around the block from: (1) opening of the release plane, allowing full hydrostatic reservoir head to penetrate to the full depth of the block; (2) dilating of block planes, allowing more water to penetrate along the planes; or (3) severing or disruption of foundation drains, impairing their ability to relieve seepage pressures.

- A post-seismic static Newmark sliding study or the 2- or 3-dimensional non-linear finite element would answer these questions.

Node 5—Dam Fails

Node 5 of the event tree considers the stability of the dam given instability of the foundation block. The movement of the block has occurred either during the earthquake (node 3) or after the earthquake (node 4). Questions that could be asked are: How stable is the dam given a certain amount of movement in the foundation? How stable is the dam given a certain lack of foundation support? Are there other 3D mechanisms that might come into play that could stabilize the dam? Is the dam too deformed or damaged to be stable? Is the dam large enough to stabilize the foundation block and prevent large releases of the reservoir?

- A 3-dimensional non-linear finite element analysis would answer these questions.

References

FEMA, 2015, “Selecting Analytic Tools for Concrete Dams to Address Key Events Along Potential Failure Mode Paths”, United States Federal Emergency Management Agency, December 2015.

CWC-Risk, 2018, “Guidelines for Assessing and Managing Risks Associated with Dams”, Doc. No. CDSO_GUD_DS_10_v1.0, Central Water Commission, July 2018.

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APPENDIX G – RECOMMENDED METHOD FOR DESIGN OF FILTERS FOR EMBANKMENT DAMS

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RECOMMENDED METHOD FOR DESIGN OF FILTERS FOR EMBANKMENT DAMS

Introduction

The recommended method for design of critical filters for embankment dams is based on IS Code 9429 (1999) as well as Sherard and Dunnigan (1985, 1989). Few minor differences between the IS Code 9429 and Sherard and Dunnigan (1985, 1989) methods are highlighted below. The Sherard and Dunnigan (1985, 1989) method has been better detailed in USBR (1987) and Fell et.al. (2005).

Filter Design Steps: the filter gradation limits are to be determined using the following steps:

Step 1: Plot the gradation curve (grain-size distribution) of the base soil materials. Use enough samples to define the range of grain sizes for the base soil or soils. Design the filter using the base soil that requires the smallest D15F size for filtering purposes. Base the design for drainage purposes on the base soil that has a representative (say median) D15B size.

Step 2: Proceed to Step 4 if the base soil contains no gravel (material larger than 4.75 mm) or if designing coarse filters, where the base soil is the fine filter.

Step 3: For broadly graded soils, which have some medium and coarse gravel, an adjusted particle size distribution should be used. As shown in example Figure G-1 below, this involves taking the particle size distribution and adjusting it to what it would be if only the fraction passing the 4.75 mm sieve were used. Prepare adjusted gradation curves for base soils that have particles larger than the 4.75 mm sieve as follows.

- Obtain a correction factor by dividing 100 by the percent passing the 4.75 mm sieve;
- Multiply the percentage passing each sieve size of the base soil smaller than 4.75mm sieve by the correction factor determined above;
- Plot these adjusted percentages to obtain a new gradation curve;
- Use the adjusted curve to determine the percentage passing the 0.075 mm sieve in Step 4.

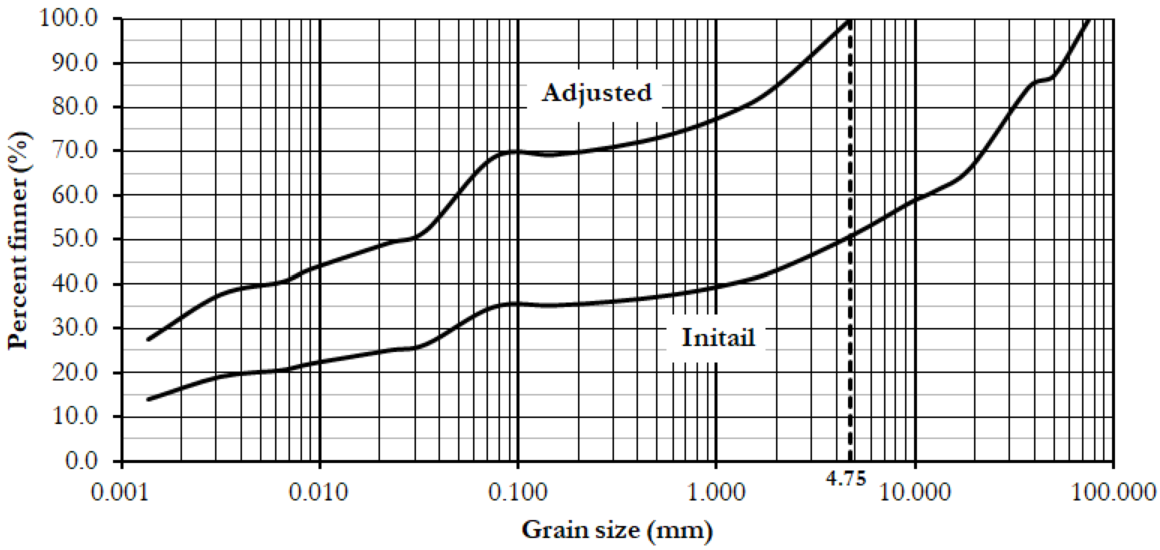

Figure G-1: Adjustment of the particle size distribution for gravelly base soils (see table below for calculation).

Sieve Size (mm)	*Percentage Passing (initial)*	*Percentage Passing (adjusted) = (100/50.9)×initial*
75.00	***100.0***	
50.00	*87.2*	
37.50	*84.7*	
19.00	*66.4*	
12.50	*61.1*	
9.50	*58.6*	
4.75	***50.9***	*100.00*
2.00	*43.2*	*84.76*
1.18	*40.1*	*78.80*
0.60	*37.7*	*73.97*
0.30	*36.2*	*71.01*
0.15	*35.3*	*69.26*
0.08	*35.0*	*68.62*
0.03	*26.5*	*52.04*
0.0219	*25.0*	*49.15*
0.0090	*22.1*	*43.37*
0.0064	*20.6*	*40.48*
0.0031	*19.1*	*37.59*
0.0014	*14.0*	*27.47*

Step 4: Place the base soil in a category determined by the percent passing the 0.075 mm sieve from the regarded gradation curve data according to Table G-1.

Table G-1: Base soil categories

Base soil category	**% finer than 0.075 mm (after regarding where applicable)**	**Base soil description**
1	> 85	Fine silts and clays
2A	35 to 85*	Silty and clayey sands; sandy clays; and clay, silt, sand, gravel mixes
4A	15 to 35**	Silty and clayey sands and gravel
3	< 15	Sand and gravels

*In IS Code 9429 the range is 40 to 85. **In IS Code 9429 the range is 15 to 39.

Step 5: To satisfy filtration requirements, determine the maximum allowable D15F size for the filter in accordance with the Table G-2.

Table G-2: Filtering criteria – maximum D15F.

Base soil category	**Filtering Criteria**
1	≤ 9×D85B but not less than 0.2 mm
2A	≤ 0.7 mm
3	≤ 4 ×D85B of base soil after regrading
4A	(35* - A/35* - 15)[(4×D85B) - 0.7 mm) + 0.7 mm. A = % passing 0.75 mm sieve after regrading (if 4×D85B is less than 0.7 mm, use 0.7 mm).

*** In IS Code 9429, the value of 40 is used instead of 35.**

Step 6: To ensure the filter is sufficiently permeable, determine the minimum allowable D15F in accordance with Table G-3. The permeability requirement is determined from the D15 size of the base soil gradation before regarding.

Table G-3: Permeability criteria

Base soil category	Filtering Criteria
All categories	Minimum D15F≥4×D15B* of the base soil before regrading, but not less than 0.1 mm; < 2% (or at most 5%) fines passing 0.075 mm sieve in the filter; fines non plastic.

*In IS Code 9429, the factor is 5 instead of 4 (i.e. D15F≥5×D15B).

Step 7: The width of the allowable filter design band must be kept relatively narrow to prevent the use of possibly gap-graded filters, but wide enough to allow manufacture. Adjust the maximum and minimum D15F sizes for the filter band determined in Steps 5 and 6 so that the ratio is 5 or less at any given percentage passing of 60 or less and adjust the limits of the design filter band so that the coarse and fine sides have a coefficient of uniformity D60/D10 of 6 or less. The use of a broad range of particle sizes to specify a filter gradation could result in allowing the use of gap-graded materials. Materials that have a broad range of particle sizes may also be susceptible to segregation during placement.

Step 8: To minimize segregation during construction, use a maximum size of 75 mm in filter zones which are not less than 2 m wide or 0.5 m thick. For narrower and thinner filter zones (particularly Zone 2A filters) use a maximum size of 37 mm or 50 mm. Consider the relationship between the maximum D90 and the minimum D10 of the filter. Calculate a preliminary D10F size by dividing the minimum D15F by 1.2. (This factor of 1.2 is based on the assumption that the slope of the line connecting D15F and D10F should be on a coefficient of uniformity of about 6.) Determine the maximum D90F using Table G-4. For Zone 2B filters, use the coarse limit D10F in Table G-4. Sand filters that have a D90F less than about 20 mm generally do not require special adjustments for the broadness of the filter band. For coarser filters and gravel zones that serve both as filters and drains, the ratio of D90F/D10F should decrease rapidly with increasing D10F sizes.

Table G-4: Segregation criteria

Base soil category	If D10F (mm) is:	Then maximum D90F (mm) is
All categories	< 0.5	20
	0.5 to 1.0	25
	1.0 to 2.0	30
	2.0 to 5.0	40
	5.0 to 10	50
	> 10	60

Step 9: Connect the control points to form a preliminary design for the fine and coarse sides of the filter band. Complete the design by extrapolating the coarse and fine curves to the 100 percent finer value. For purposes of writing specifications, select appropriate sieves and corresponding percent finer values that best reconstruct the design band and tabulate the values.

ILLUSTRATIVE EXAMPLE

Grain size laboratory test on samples of a clay core gave the results summarized in Table G-5 below. Design fine and coarse filter for this clay core to be used in a zoned earth fill dam.

Table G-5: gradation test results on samples of a clay core material.

Sieve Size (mm)	*Percentage Passing*	*Percentage Passing*	*Percentage Passing*	*Percentage Passing*	*Percentage Passing*
2	*100.00*	*100.00*	*100.00*	*100.00*	*100.00*
1.18	*100.00*	*100.00*	*100.00*	*100.00*	*99.30*
0.6	*99.86*	*99.95*	*99.98*	*99.98*	*97.73*
0.3	*99.48*	*99.68*	*99.95*	*99.73*	*96.41*
0.15	*98.84*	*99.07*	*99.89*	*98.46*	*95.37*
0.075	*98.11*	*98.21*	*99.66*	*96.52*	*94.39*
0.0316	*88.23*	*80.63*	*85.23*	*61.58*	*83.54*
0.0203	*82.19*	*71.28*	*73.40*	*56.08*	*78.25*
0.0121	*72.52*	*64.27*	*61.56*	*49.48*	*71.91*
0.0087	*66.47*	*57.26*	*56.82*	*46.18*	*63.45*
0.0062	*64.06*	*54.92*	*50.90*	*40.68*	*59.22*
0.0031	*55.60*	*42.07*	*41.43*	*35.19*	*45.47*
0.0013	*49.55*	*35.06*	*31.96*	*29.69*	*31.72*

DESIGN OF FINE FILTER (F1)

Step 1: Plot the gradation curve (grain-size distribution) of the base soil materials. The gradation curves are plotted as shown in Figure G-2.

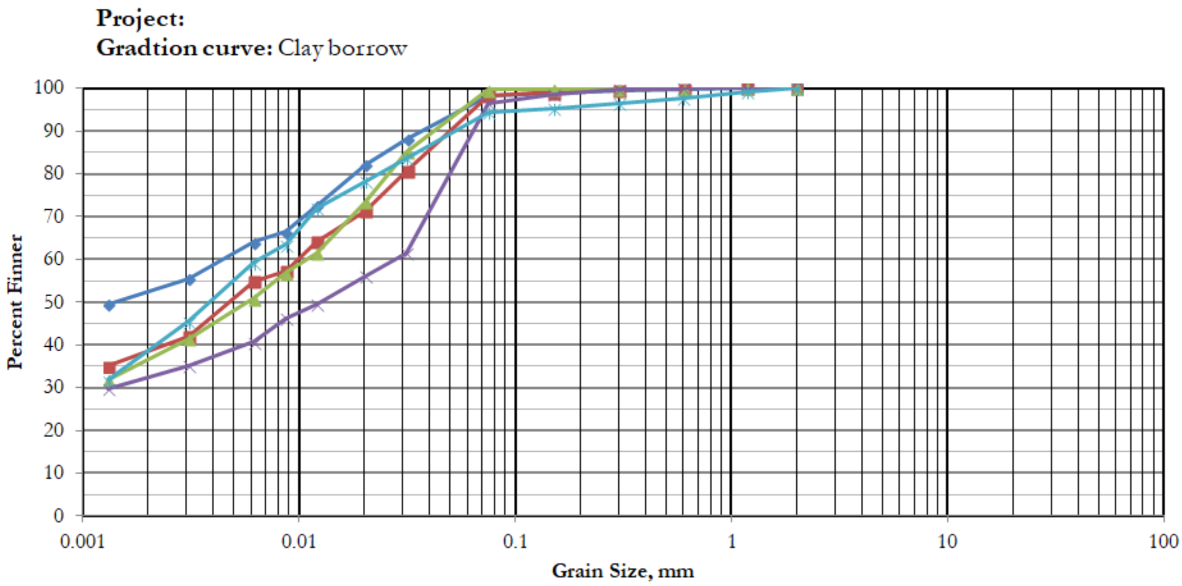

Figure G-2: gradation curves of the impervious clay core samples.

Step 2: Proceed to Step 4 as none of the samples contain gravel.

Step 4: According to Table G-1 and IS Code 9429, base soil is in **category 2A**.

Step 5: Based on Table G-2 and IS Code 9429 maximum D15F should be < 0.7 mm. Take max **D15F = 0.4 mm**.

Step 6: Based on Table G-3, minimum D15F should be ≥ 0.1 mm and in addition < 2% (or utmost 5%) fines passing 0.075 mm sieve. Take min D15F = **0.12 mm**.

Step 7: $\frac{\text{D15F}_{\max}}{\text{D15F}_{\min}} = \frac{\mathbf{0.4}}{\mathbf{0.12}} = \mathbf{3.33 < 5}$, OK!

Step 8: Min D10F = min D15F / 1.2 = 0.12 / 1.2 = **0.10 mm**.

Max D10F = max D15F/1.2 = = 0.4 / 1.2 = **0.33 mm.**

To prevent the use of possible gap-graded filters, C_u (Coefficient of Uniformity) = D60F/D10F ≤ 6

Max D60F = 6×0.33 = **2.0 mm**

Min D60F = 6×0.10 = **0.6 mm**

For Min D10F = 0.10 mm (< 0.5 mm), take Min D90F = **2.0 mm**

For Max D10F = 0.33 mm (< 0.5 mm), take Max D90F = **7.0 mm**

Step 9: Connect the control points. Points will be connected after designing coarse filter (F2)

Design of Coarse Filter (F2)

Step 1: Use F1 filter as the base material.

Step 2: Proceed to Step 4 for design of coarse filter (F2).

Step 4: Plot gradation for F1 filter (see Figure G-3 below). Based on its gradation, IS 9429 and Table G-1, the F1 filter in **category 3**.

Step 5: According to Table G-2 and IS 9429, a suitable maximum filter (for filtering criteria) is:

Max D15F ≤ 4×D85B (Min D85F of base = 1.6 mm from the curve)

Max D15F = 3.5×1.6 mm (note that 3.5 < 4)

Max D15F = **5.6 mm**

Step 6: The width of the allowable filter design band must be kept relatively narrow to prevent the use of possible gap-graded filters. The ratio between the Max D15F and Min D15F should be 5 or less. Taking a ratio of 3.5:

D15F (min) = D15F (min)/3.5

D15F (min) = 5.6/3.5

D15F (min) = 1.6 mm

Step 7: Also according to Table G-3 Minimum D15F≥4×Max D15B (for permeability criteria). Max D15B = 0.4 mm and 4×Max D15B = 1.6 mm, OK!

Step 8: Min D10F = Min D15F / 1.2 = 1.6 / 1.2 = **1.33 mm**.

Max D10F = max D15F/1.2 = = 5.6 / 1.2 = **4.67 mm.**

To prevent the use of possible gap-graded filters, C_u (Coefficient of Uniformity) = D60F/D10F ≤ 6

Max D60F = 5×4.67 = **23.33 mm.**

Min D60F = 5×1.33 = **6.67 mm.**

According to Table G-4 and IS 9429,

For Min D10F = 1.33 mm (1.0 to 2.0 mm), Max D90F = 30 mm, take **15 mm**.

For Max D10F = 4.67 mm (2 to 5 mm), take Max D90F = **40.0 mm.**

Step 9: Connect the control points. All points for filter F1 and filter F2 are connected and shown in Figure G-3 below. Table F-6 gives the grading limits based on Figure G-3 and standard and similar sieve sizes. Such table is finally carried to Technical Specifications.

Table: G-6: Grading Limits for F1 and F2 filters.

Particle size (mm)	**F1 Filter**		**F2 Filter**	
	% finer (Fine Limit)	**% finer (Coarse Limit)**	**% finer (Fine Limit)**	**% finer (Coarse Limit)**
75	-	-	-	-
50	-	-	-	100
30	-	-	-	75
25	-	-	-	62
20			100	55
15	-	-	90	45
10	-	100	75	32
6	-	86	57	17
5.25	-	83	52	13
4	-	76	45	6
3.15	-	71	36	0
3	100	70	35	-
2	90	60	21	-
1.5	82	50	10	-
0.9	70	37	0	
0.6	60	26	-	-
0.27	45	4	-	-
0.23	33	0		
0.075	2	-	-	-
0.07	0	-	-	-

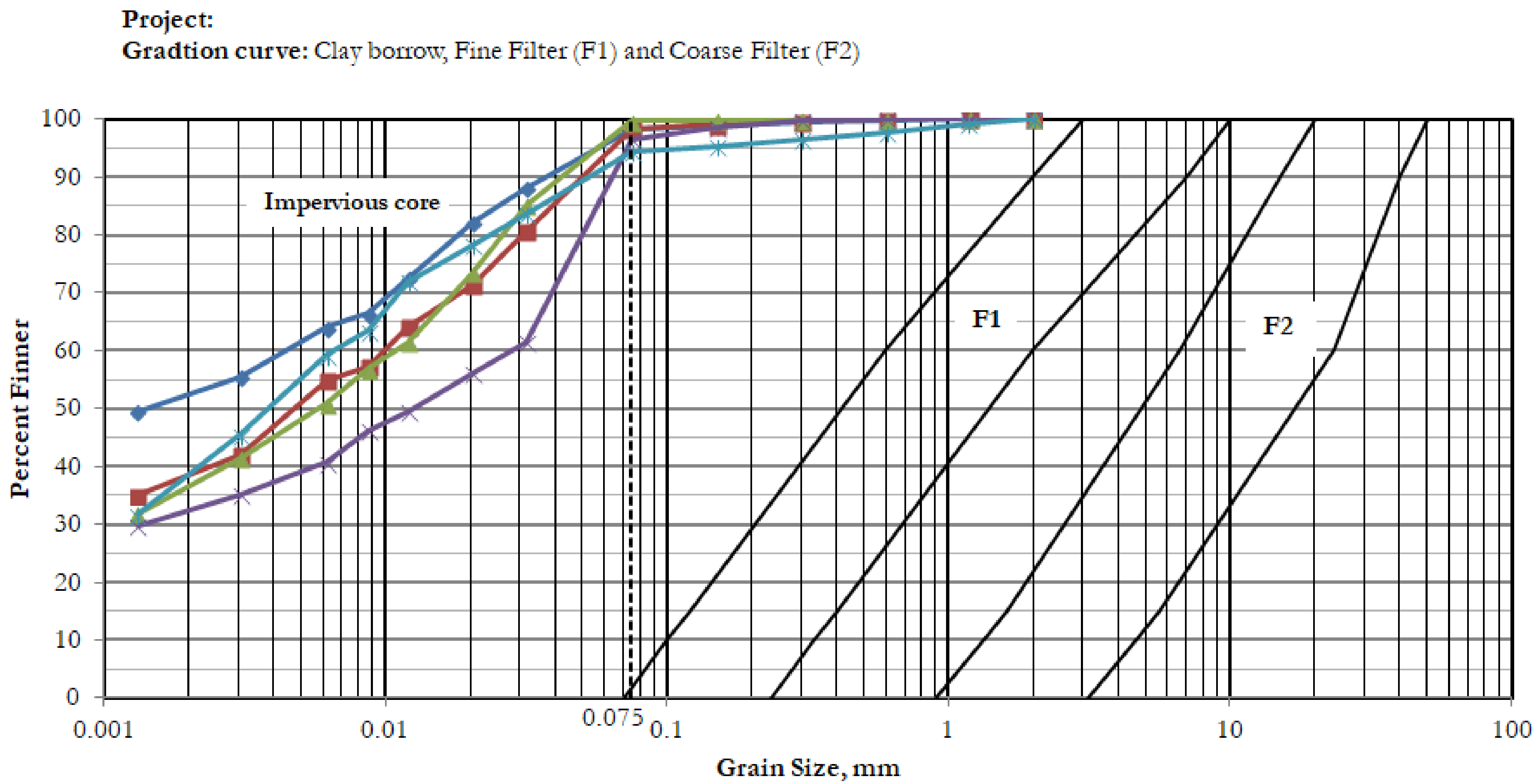

Figure G-3: designed Fine filter (F1) and Coarse filter (F2) for an impervious clay core.

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APPENDIX H – METHOD FOR DESIGN OF UPSTREAM SLOPE PROTECTION RIP RAP FOR EMBANKMENT DAMS (USBR)

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METHOD FOR DESIGN OF UPSTREAM SLOPE PROTECTION RIP RAP FOR EMBANKMENT DAMS (USBR)

IS 8237 (1985) and USBR standard No. 13 (chapter 7) are generally used for the design of riprap. The USBR method provides more details on how to calculate the required size and weight of rock as well as riprap thickness. This method is illustrated below and an example is provided.

Step 1: determine the weight of the rock in the riprap where 50% is smaller using the following equation.

$$W_{50} = \frac{\gamma_r H_s^3}{4.37(G_s - 1)^3(\cot\alpha)} \quad \textbf{(lb)} \qquad \textbf{Tolerable Damage}$$

$$W_{50} = \frac{\gamma_r H_s^3}{3.62(G_s - 1)^3(\cot\alpha)^{2/3}} \quad \textbf{(lb)} \qquad \textbf{Zero damage}$$

where,

W_{50} = weight of the rock in riprap where 50% is smaller (lb).

$\boldsymbol{\gamma_r}$ = specific unit of the rock (lb/ft^3) - to be determined by laboratory test on rock samples.

H_s = significant wave height (ft) - to be determined as per acceptable standards.

G_s = specific gravity of the rock - to be determined by laboratory test on rock samples.

Step 2: determine the maximum and minimum weights using the following equations, respectively:

$$W_{max} = 4 \times W_{50}$$

$$W_{min} = W_{50}/8$$

where, W_{max} = 100% of the rock in the riprap is smaller.

W_{min} = approximately 5% of the rock in the riprap is smaller.

Step 3: convert from rock weights to representative diameter using the following equation:

$$\textbf{VOL} = 0.75 \times D_n^3$$

where, VOL = rock volume = $Wn/\boldsymbol{\gamma_r}$.

W_n = weight of rock in the riprap where n% is smaller

D_n = representative size of rock in the riprap where n% is smaller.

Step 4: determine thickness of the riprap using the following equation:

$$\boldsymbol{T = 20(W_{50}/\gamma_r)^{0.333}}$$

where, T = riprap thickness (inches).

Step 5: prepare riprap band curves using the above data and the following guidelines (see Figure 1).

- Start with a band where the W_{50} defines the D_{55} for the fine limit and D_{35} for the coarse limit

- Start with a band where the W_{max} defines the D_{100} for the coarse limit. For the D_{100}, a practical band width will be one in which the limits are separated by at least 20 percent.
- Start with a band where the W_{min} defines the D_5 for the coarse limit and the D_{25} for the fine limit.

Note- USBR recommends to use tolerable damage condition if damage to the riprap does not cause dam failure and periodic repair of damaged riprap is justified.

ILLUSTRATIVE EXAMPLE

Riprap computation (Design standard 13, Chapter 7, USBR)

γ_r	Specific unit weight	137.3416	lb/ft³	2200	kg/m³
H_s	Significant wave height	3.28	ft	1	m
G_s	Specific gravity of rock	2.65		2.65	
α	Slope angle of u/s slope	26.5651	^{0}	26.5651	^{0}
cot(α)	2.0 (1V:2H upstream slope)				
W_{50}	Weight of the rock in the riprap where 50% is finer				
$W_{50} = \dfrac{\gamma_r H_s^3}{4.37(G_s-1)^3(\cot\alpha)}$ (lb)				Tolerable damage	
$W_{50} = \dfrac{\gamma_r H_s^3}{3.62(G_s-1)^3(\cot\alpha)^{2/3}}$ (lb)				Zero damage	

W_{50}	123.54	lb	56.03	kg	Tolerable damage
W_{50}	324.68	lb	147.27	kg	Zero damage
$W_{100} = 4\times W_{50}$	494.14	lb	224.14	kg	Tolerable damage
$W_{100} = 4\times W_{50}$	1298.72	lb	589.08	kg	Zero damage
$W_{min} = W_{50}/8$	15.44	lb	7.0	kg	Tolerable damage
$W_{min} = W_{50}/8$	40.58	lb	18.41	kg	Zero damage

VOL = $0.75\times D_n^3$					
VOL = rock volume = W_n/γ_r		W_n = weight where n% is finer		D_n = size n% is finer	
D_{50}	1.06	ft	0.32	m	Tolerable damage
D_{50}	1.47	ft	0.45	m	Zero damage
D_{100}	1.69	ft	0.51	m	Tolerable damage
D_{100}	2.33	ft	0.71	m	Zero damage
D_{min}	0.53	ft	0.16	m	Tolerable damage
D_{min}	0.73	ft	0.22	m	Zero damage

Riprap layer thickness* $T = 20(W_{50}/\gamma_r)^{0.333}$ (inches)				
T = 19.31	in	T = 0.5	m	Tolerable damage
T = 26.63	in	T = 0.7	m	Zero damage

*This thickness is perpendicular to the slope.

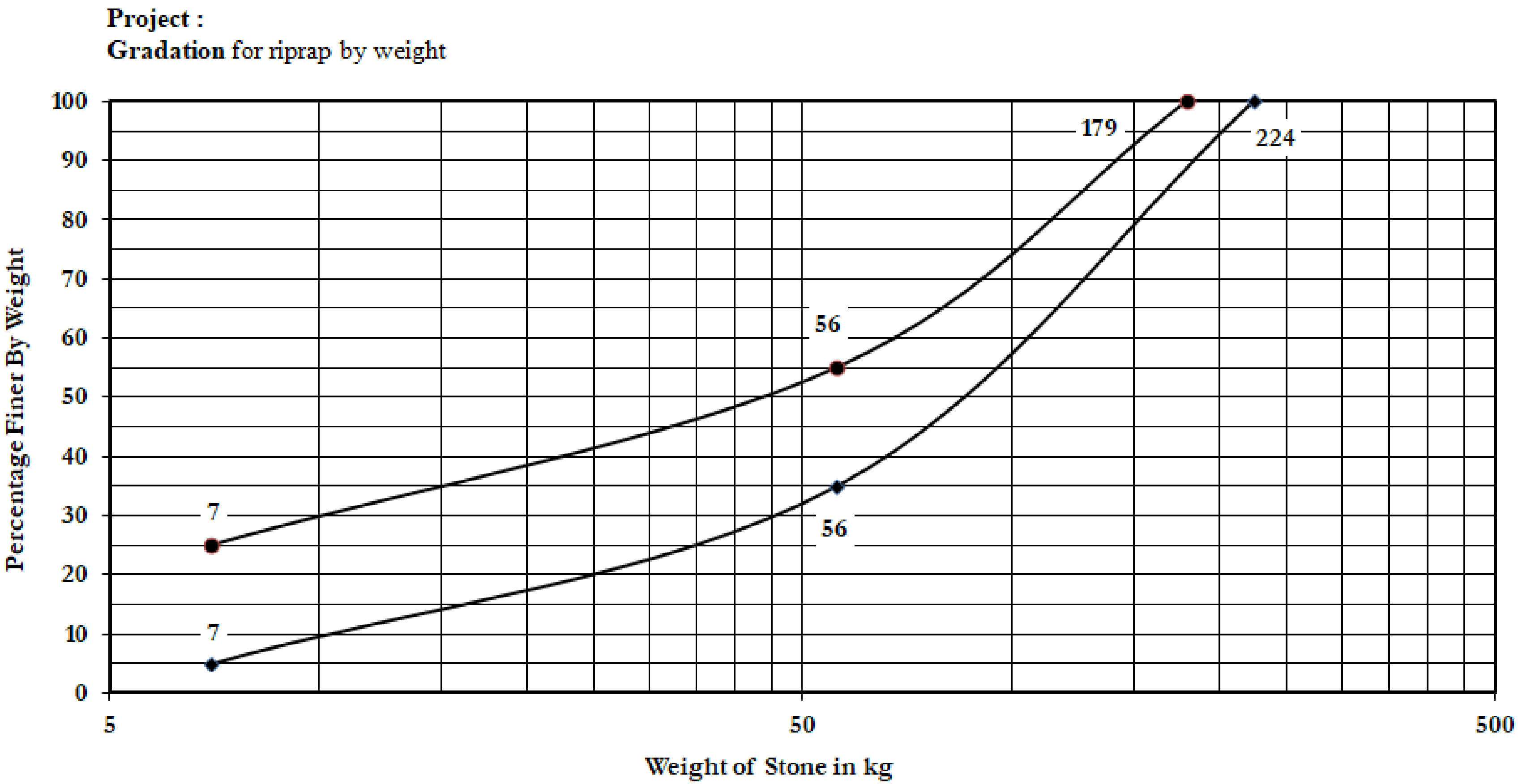

Figure 1: Riprap gradation by weight (tolerable damage).

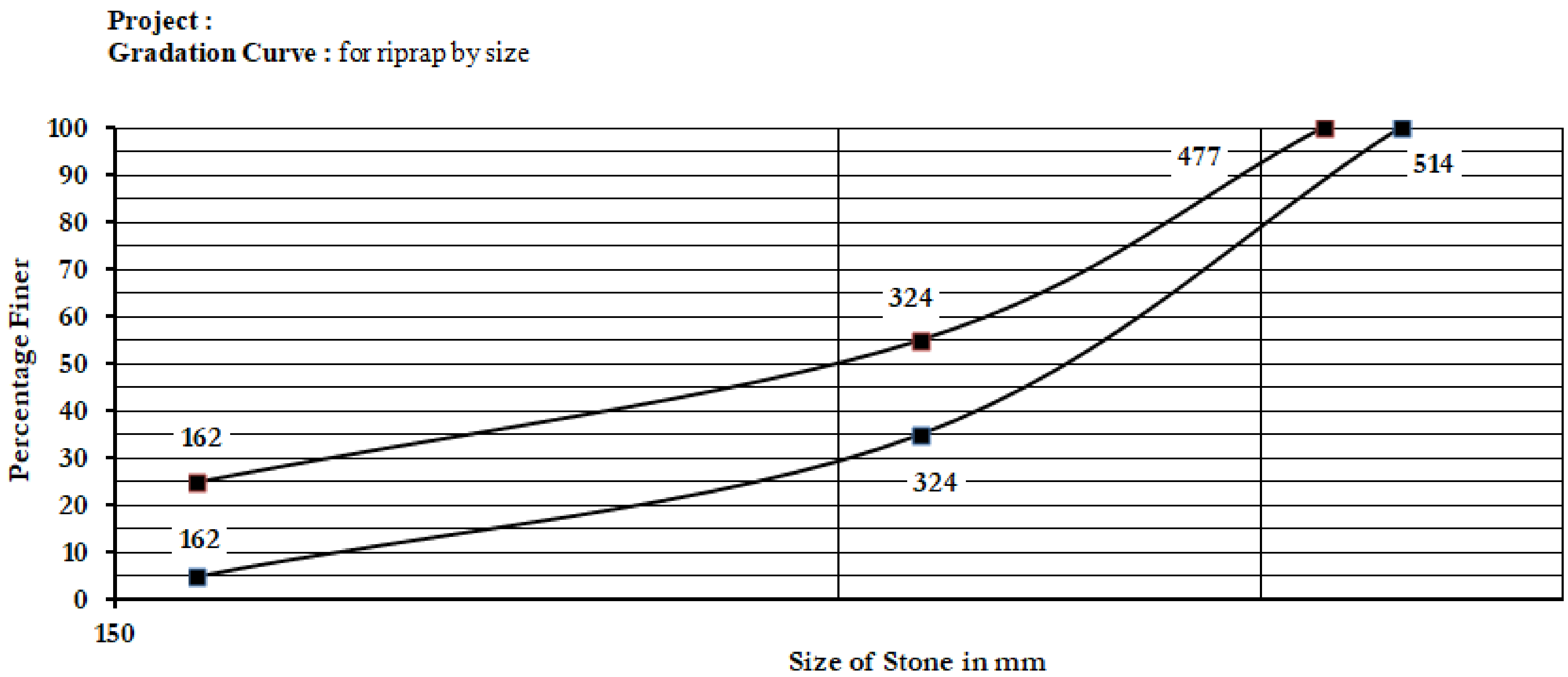

Figure 2: Riprap gradation by size (tolerable damage).

APPENDIX I – APPROXIMATE METHOD FOR DETERMINATION OF PHREATIC SURFACE

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APPROXIMATE METHOD FOR DETERMINATION OF PHREATIC SURFACE

To determine the phreatic surface for seepage through embankment dams, current methods use Finite Element Method (FEM) based seepage analysis using state of the art software such as SEEP/W from GeoSlope. In this section, Casagrande's approximate method of drawing the phreatic surface for seepage through an earthfill dam is described below (the procedure is adapted from the book *Soil Mechanics and Foundations*, Budhu, M., 3rd edition, 2011). Casagrande showed that the phreatic surface can be approximated by a parabola with corrections at the points of entry and exit.

The focus (origin) of the parabola is at the toe of the dam, point **F** (see Figure 1).

The procedure to draw a phreatic surface within an earthfill dam, with reference to Figure 1, is as follows.

1. Draw the structure to scale.

2. Locate a point **A** at the intersection of a vertical line from the bottom of the upstream face and the water surface, and a point **B** where the waterline intersects the upstream face.

3. Locate point **C**, such that BC = 0.3×AB.

4. Project a vertical line from **C** to intersect the base of the dam at **D**.

5. Locate the focus of the basic parabola. The focus **F** is located at the toe of the dam.

6. Calculate the focal distance using the following equation,

$$f = (\sqrt{b^2 + H^2} - b)/2$$

where **b** is the distance **FD** and **H** is the height of water at the upstream face.

7. Construct the basic parabola using the following equation:

$$z = 2\sqrt{f(f + x)}$$

8. Sketch in a transition section **BE**.

9. Calculate the length of the discharge face, **a**, using:

$$\boldsymbol{a} = \frac{\boldsymbol{b}}{\cos\boldsymbol{\beta}} - \boldsymbol{cos\beta}\sqrt{\boldsymbol{b^2 - H^2 cot^2\beta}}; \quad \boldsymbol{\beta \leq 30^0}$$

For $\beta > 30^0$, use Figure F-2 and

- **a)** measure the distance **TF**, where **T** is the intersection of the basic parabola with the downstream face;
- **b)** for the known angle β, read the corresponding factor $\Delta a/L$ from the chart; and
- **c)** find the distance $a = TF(1 - \Delta a/L)$.

10. Measure the distance **a** from the toe of the dam along the downstream face to point **G**.

11. Sketch in a transition section, **GK**.

12. Calculate the flow using:

$$q = ak \times sin\beta \times tan\beta$$

where **k** is the coefficient of permeability. If the downstream slope has a horizontal drainage blanket as shown in Figure 3, the flow is calculated using: $q = 2fk$.

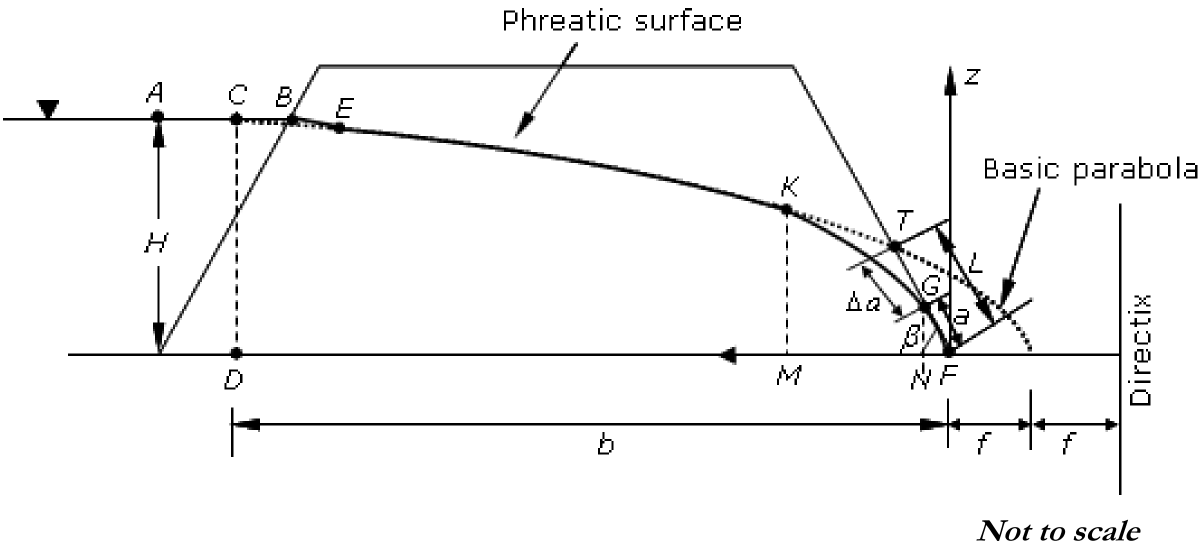

Figure 1: Phreatic surface within an earth dam.

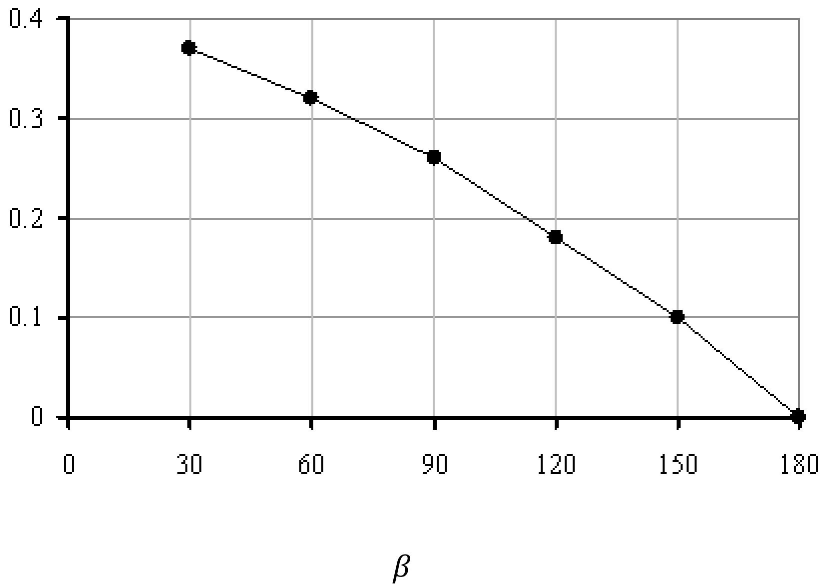

Figure 2: Correction factor for downstream curve.

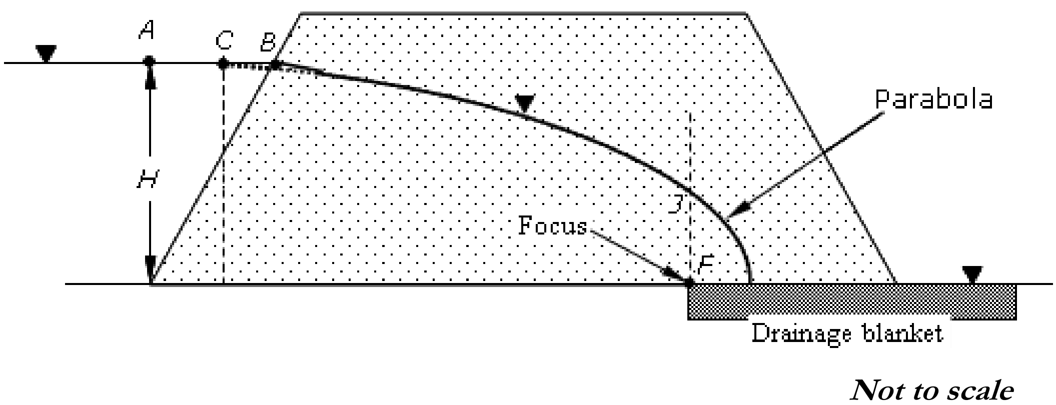

Figure 3: A horizontal drainage blanket at the toe of an earth dam.

APPENDIX J – AN EXAMPLE OF EMBANKMENT DAM STABILITY ANALYSIS USING GEOSLOPE SOFTWARE

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AN EXAMPLE OF EMBANKMENT DAM STABILITY ANALYSIS USING GEOSLOPE SOFTWARE

1. Introduction

This Appendix provides an example of a stability analysis of a zoned embankment dam, 22 m high. The foundation of the dam consisted of weak organic material that needed to be removed, resulting in deep foundation excavation. The stability analysis was carried out using state-of-the art computer program SLOPE/W from GeoSlope International.

Methods, material properties and results of the analysis are summarized below.

The stability analysis was conducted in order to determine the factors of safety for various slip surfaces of:

- Upstream and downstream slopes under steady state seepage condition with and without earthquake.
- Upstream slope under sudden drawdown condition.
- Upstream and downstream slopes during and end of construction conditions (**this does not apply for existing dams but included here for general reference**).

Figure 1 provides typical cross-section of the dam and Figure 2 provides details of the dam zoning. Figure 2 is prepared using the Geoslope software.

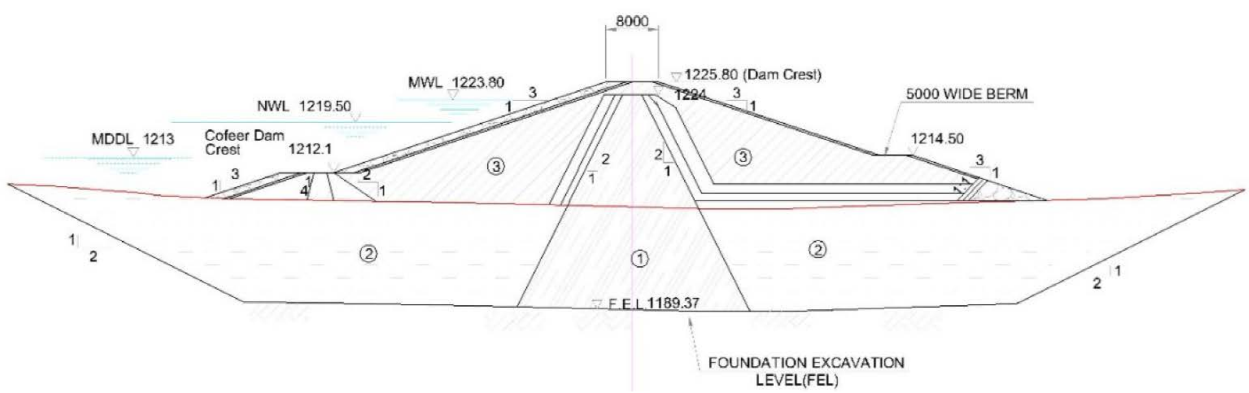
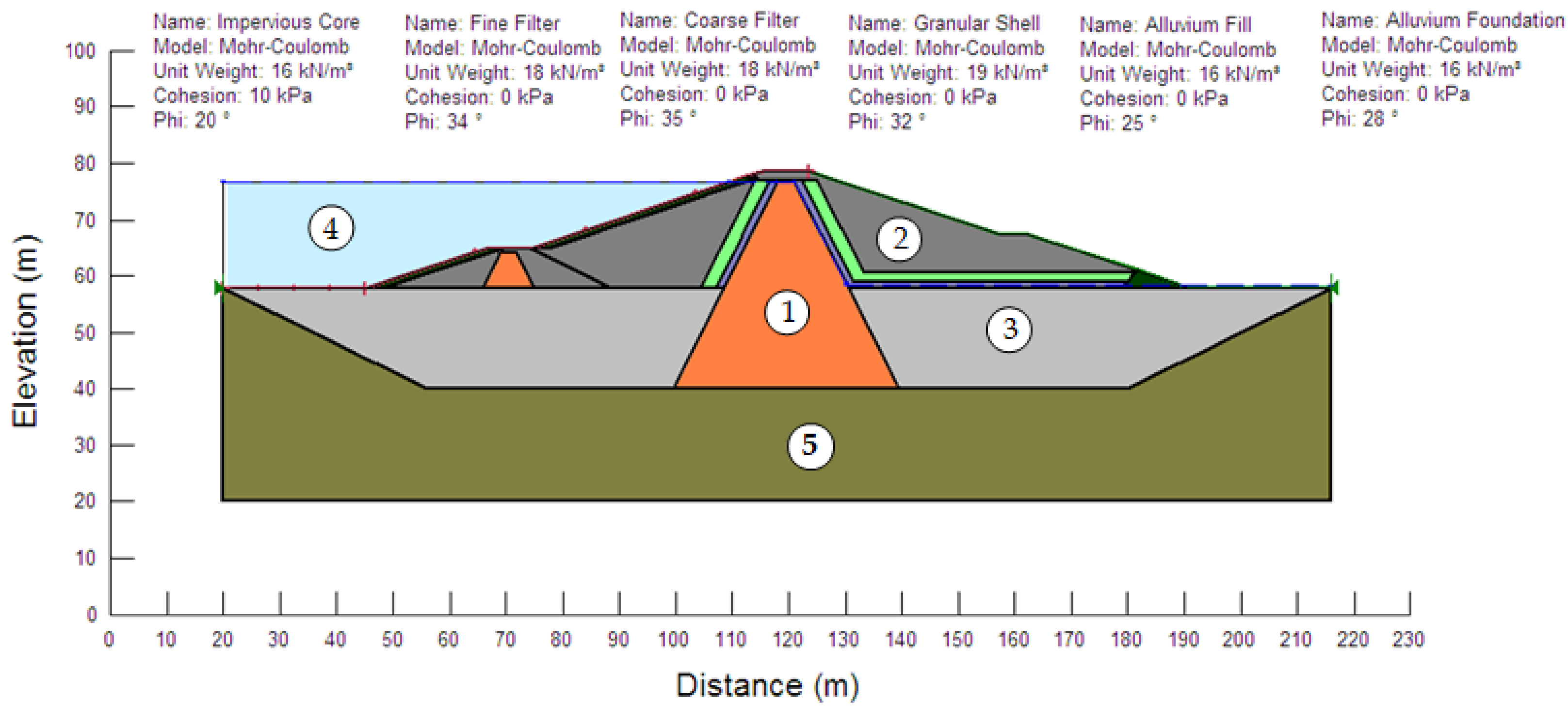


Figure 1: Typical dam cross-section.

Based on the stability analysis results shown below, the stable slopes for the proposed dam under all loading conditions are upstream slope of 1V:3H and downstream slope of 1V:3H. The impervious core has a slope of 0.5H:1V on both sides.

2. Loading Conditions

Table 1 below summarizes the loading conditions and corresponding minimum factor of safety (FOSmin) requirements proposed by USACE and used worldwide. The dam was designed to meet these requirements.



Zone 1: impervious core **Zone 2**: compacted gravel fill
Zone 3: compacted alluvium **4**: water
5: alluvial foundation

Figure 2: Dam Zoning.

Table 1: Loading conditions.

Case	Loading Condition	Slope	FOS_{min}
I	During construction	Upstream Downstream	1.3 1.3
II	End of construction	Upstream Downstream	1.3 1.3
III	Sudden drawdown	Upstream -	1.3 -
IV	Steady state seepage	Upstream Downstream	1.5 1.5
IV	Steady state seepage with earthquake	Upstream Downstream	1.1 1.1

Steady State Seepage Condition

The stability analysis for both upstream and downstream slopes under steady state condition has been checked by considering FRL for both normal loading condition and with earthquake loading condition. The phreatic surface computed with the help of Seep/W from Geoslope was used to set up the pore water pressure line in the stability analysis. The design horizontal earthquake acceleration for pseudostatic analysis is taken to be 0.15g based on geological/geotechnical report of the specific site. As per international practices, the vertical acceleration was considered as 2/3 of the horizontal acceleration, which is 0.1g. Hence, the vertical coefficient of acceleration is 0.1.

Sudden Drawdown Condition

Sudden drawdown stability computations were performed for the upstream slope for conditions occurring when the water level adjacent to the slope is lowered rapidly. For analysis purposes, it was assumed that drawdown is very fast, and no drainage occurs in materials with low permeability. For this specific dam, no drainage is assumed in the impervious core during sudden drawdown. Free drainage is assumed in the gravel shell, rip-rap and filter zones.

During and End of Construction Conditions (not required for existing dams but included here for reference purpose only).

Computation of stability during and at the end of construction was performed using drained strengths in free-draining materials. For materials that drain slowly two alternatives can be used: i) total stress analysis with undrained strengths and zero pore water pressure or ii) effective stress analysis modeling partially saturated condition with pore water pressure.

For this dam, the second alternative was used with pore water pressure during construction being higher than at the end of construction. Table 2 below summarizes the values of pore water pressure ratio r_u used for the stability analyses during and at the end of construction conditions.

Table 2: Pore Water Pressure Ratio r_u Values for Construction Conditions.

Material Zone	Pore-water pressure ratio (r_u) value	
	During construction	End of construction
Impervious core	0.50	0.40
Fine filter/ Coarse Filters	0.00	0.00
Granular shell	0.00	0.00
Alluvium fill	0.40	0.35
Alluvium foundation	0.40	0.35

3. Material Properties

The material properties used for the stability analysis of different zones of the dam were obtained from laboratory tests and summarized in Table 3 below.

Table 3: Material properties.

Material	γ (kN/m^3)	ϕ' (^{0})	c' (kPa)
Impervious core	16	20	10
Fine filter	18	34	0
Coarse filter	18	35	0
Granular shell	19	32	0
Alluvium fill	16	25	0
Alluvium foundation	16	28	0

4. Method of Stability Analysis

The slope stability investigation was carried out using the Slope/W computer program based on the limit equilibrium method and the Morgenstern-Price method was used to obtain the factors of safety. This particular method was adopted because, unlike Swedish or Bishop's or Janbu's methods, the Morgenstern-Price method satisfies both the force and moment equilibrium conditions. Spencer's method also satisfies both moment and force equilibriums and gives factors of safety values very close to those obtained by the Morgenstern-Price method.

5. Stability Analysis Results

The minimum required and computed factors of safety against slope failures under the different loading conditions for the dam are summarized in Table 4. As can be seen from Table 4, the proposed dam section is stable under all loading conditions. It has been found that the steady state seepage with earthquake loading condition is the most critical.

Based on the stability analyses results, the stable slopes for the proposed embankment dam under all loading conditions are upstream slope of 1V:3H and downstream slope of 1V:3H. The impervious core has a slope of 0.5H:1V on both sides.

Table 4: Stability Analysis Results.

Loading condition	FoS_{min}	Computed FoS	
		U/S	D/S
During construction	1.3	1.436	1.415
End of construction	1.3	1.522	1.511
Sudden drawdown	1.3	1.545	-
Steady state seepage	1.5	1.998	1.582
Steady state seepage with earthquake	1.1	1.053 ≈ 1.1	1.159

The computed critical slip surfaces corresponding to the factors of safety in Table 4 are shown in Figures 3 to 11 below.

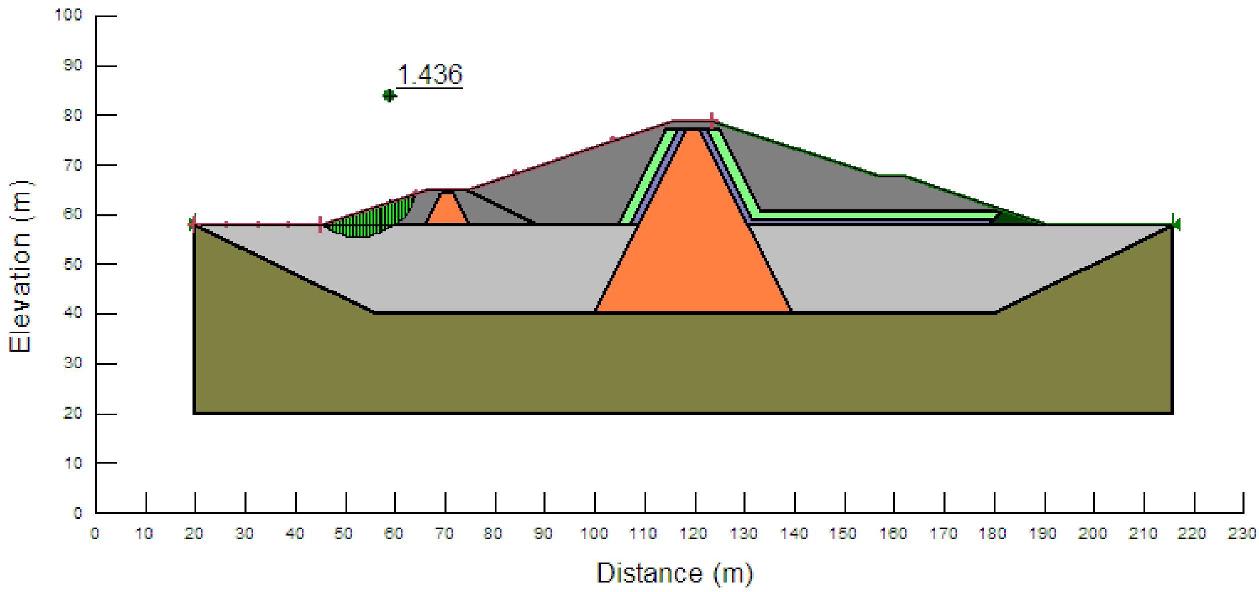


Figure 3: During construction condition (Upstream slope).

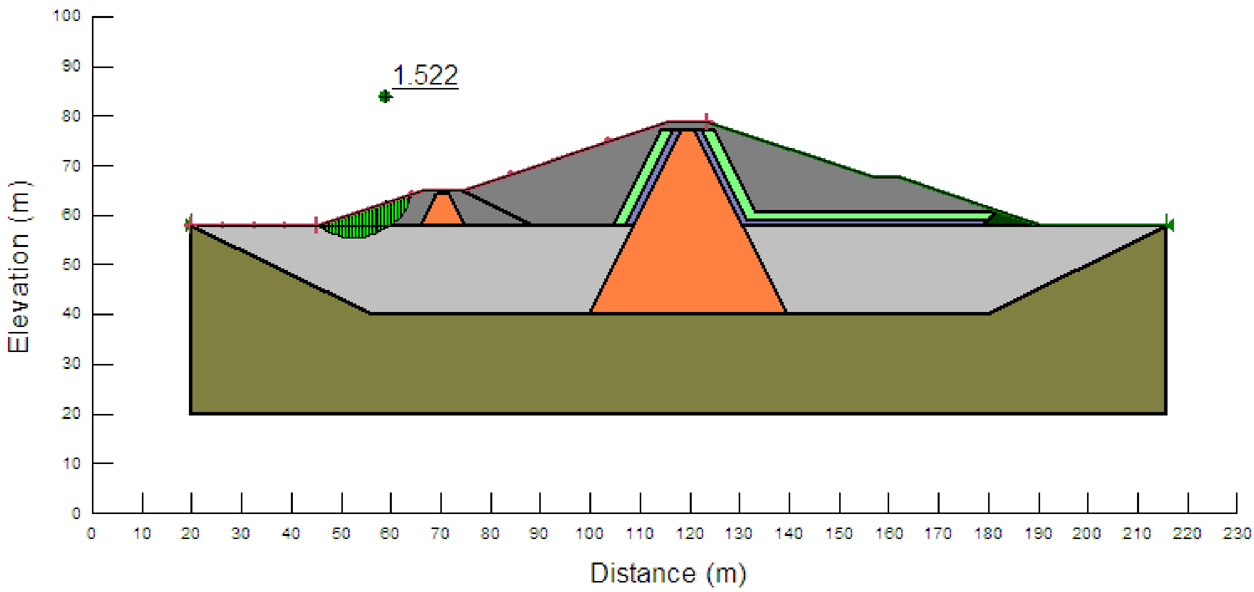


Figure 4: End of construction condition (Upstream slope).

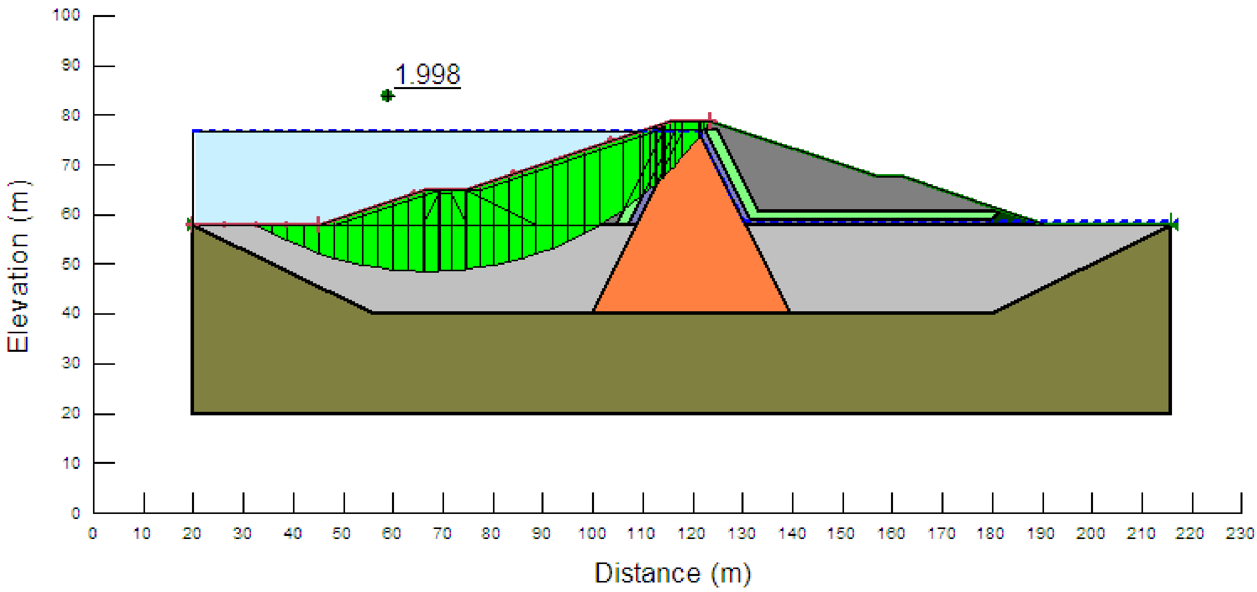


Figure 5: Steady state seepage without earthquake condition (Upstream slope).

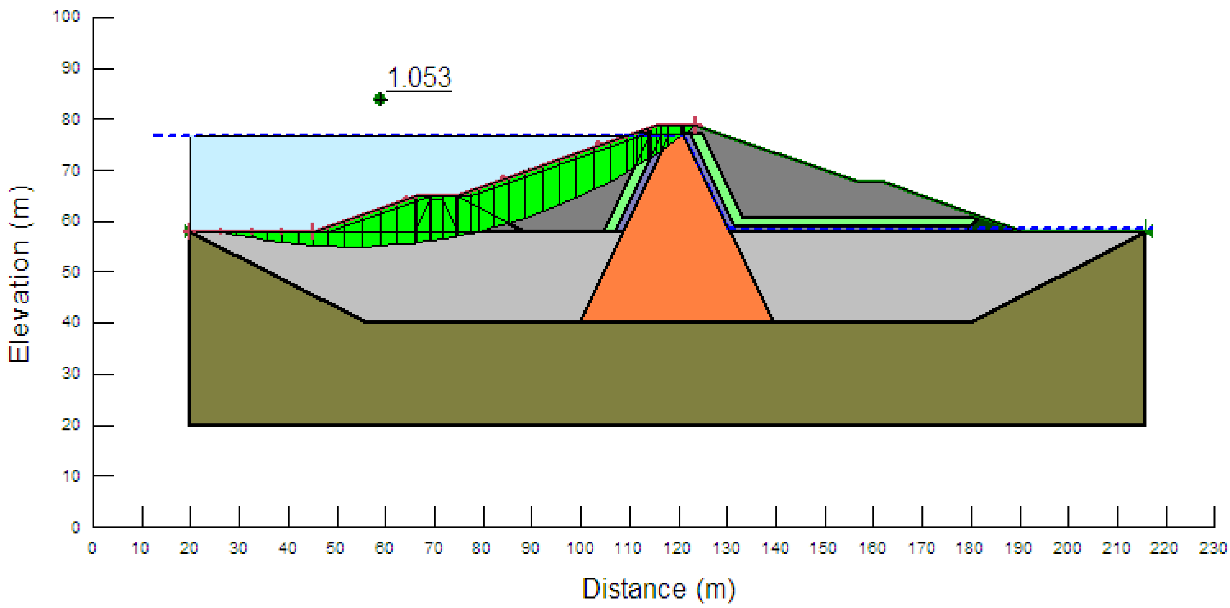


Figure 6: Steady state seepage with earthquake condition (Upstream slope).

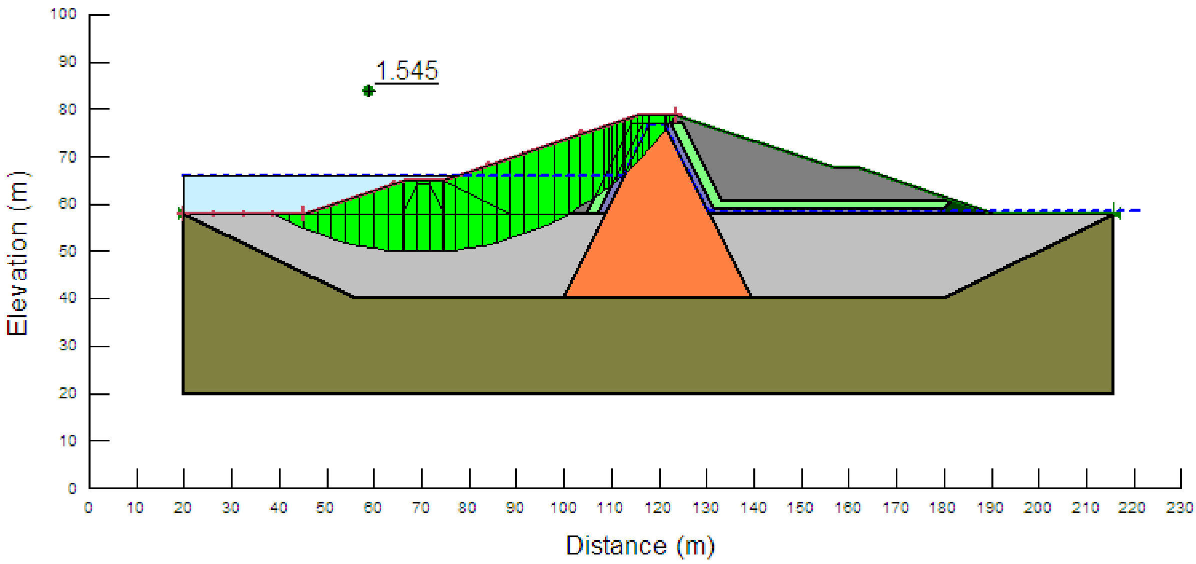


Figure 7: Sudden drawdown condition (Upstream slope).

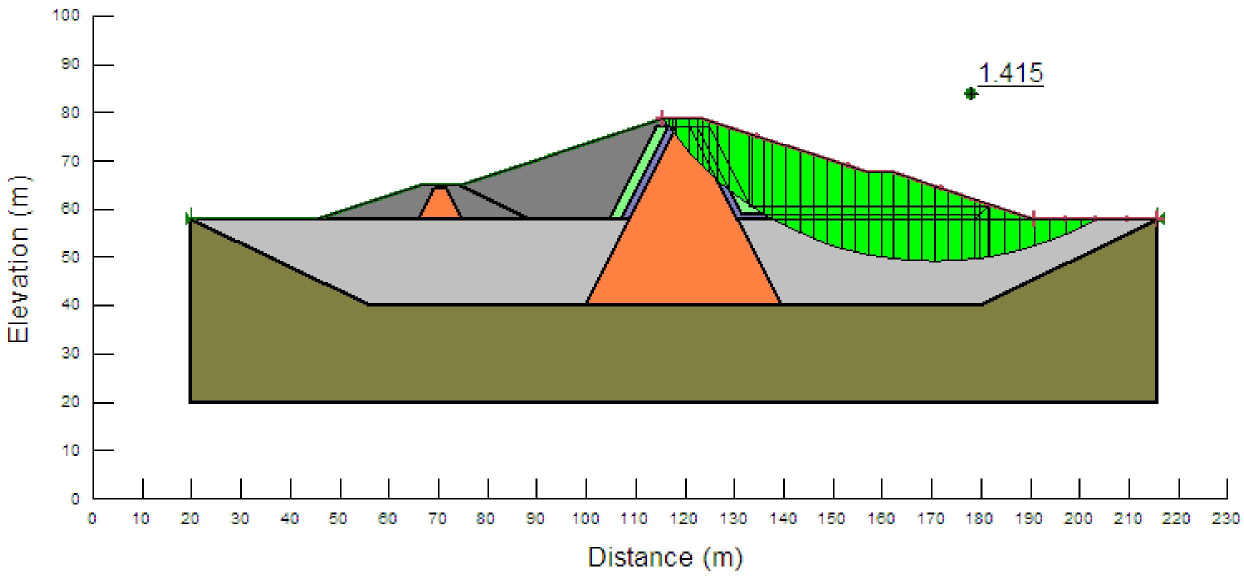


Figure 8: During construction condition (Downstream slope).

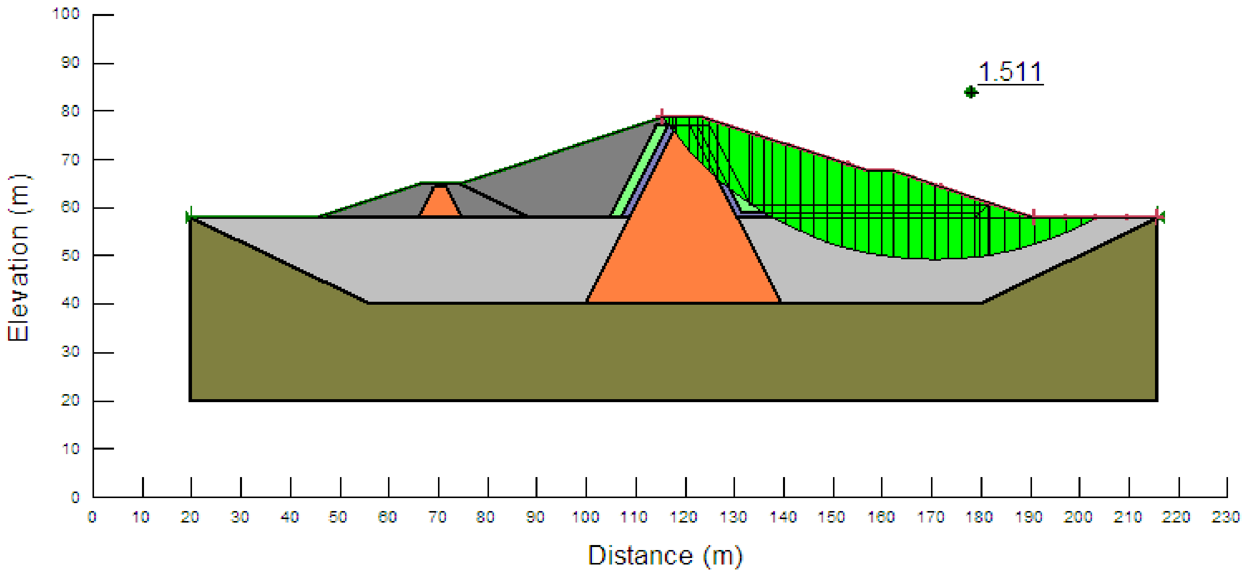


Figure 9: End of construction condition (Downstream slope).

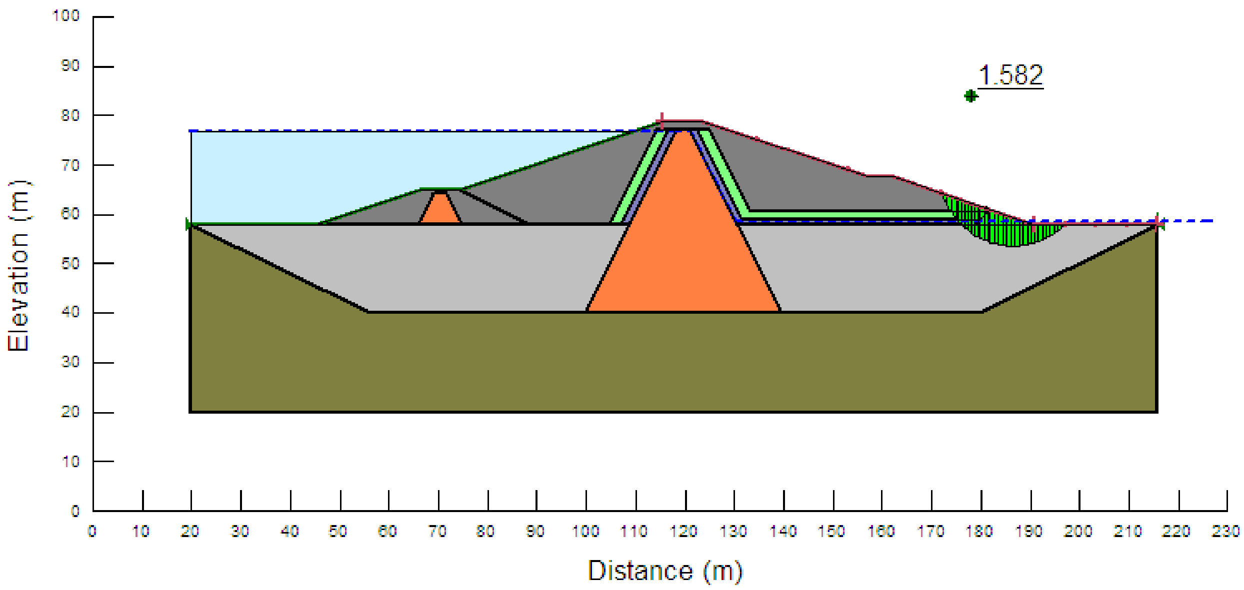


Figure 10: Steady state seepage without earthquake condition (Downstream slope).

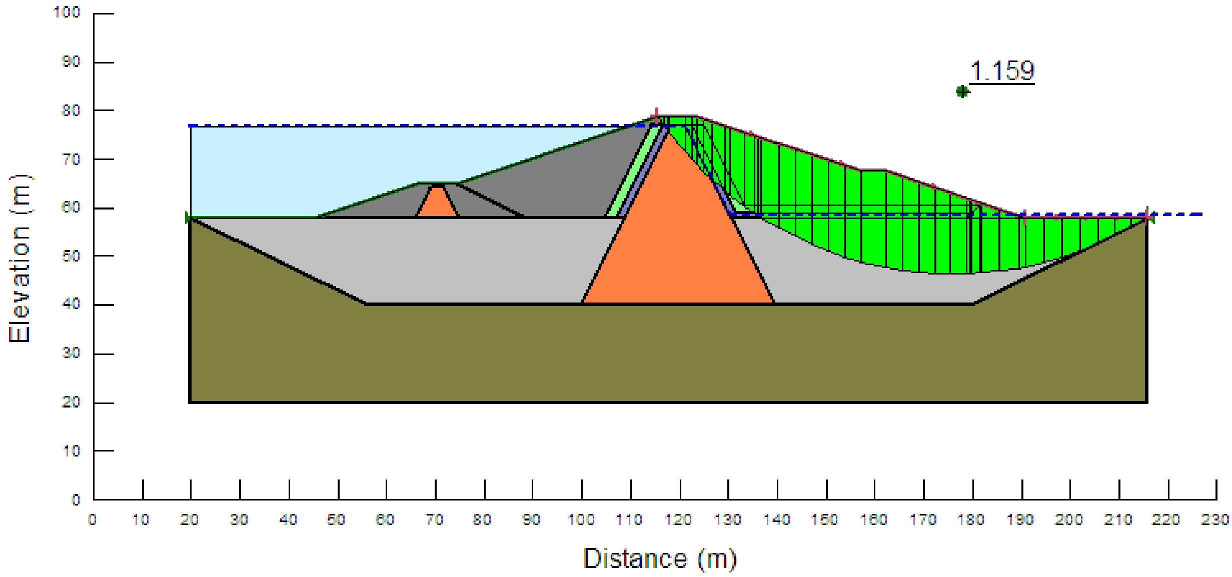


Figure 11: Steady state seepage with earthquake condition (Downstream slope).

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APPENDIX K - GLOSSARY

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GLOSSARY OF TERMS

The purpose of this glossary is to define a common vocabulary of terms for use within and among Central and State Government agencies, dam owners and operators, consulting engineers, and construction contractors. Terms have been included that are generic and apply to all dams, regardless of size, owner, or location.

Abutment – The part of the valley side against which the dam is constructed. The left and right abutments of a dam are defined with the observer looking downstream from the dam.

Appurtenant works – Structures associated with the dam including the following:

a) Spillways, either in the dam or separate therefrom;
b) Reservoir and its rim;
c) Low-level outlet works and water conduits such as tunnels, pipelines or penstocks, either through the dam or its abutments or reservoir rim;
d) Hydro-mechanical equipment including gates, valves, hoists, and elevators;
e) Energy dissipation and river training works; and
f) Other associated structures that act integrally with the dam body.

Auxiliary spillway – Any secondary spillway that is designed to be infrequently operated, in anticipation of some degree of structural damage or erosion to the spillway that would occur during operation.

Barrage – While the term barrage is borrowed from the French word meaning "dam" in general, its usage in English refers to a type of low-head, dam that consists of many large gates that can be opened or closed to control the amount of water passing through the structure, and thus regulate and stabilize river water elevation upstream for diverting flow for irrigation and other purposes.

Berm – A horizontal part of the slope of an embankment or cutting.

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.

Breach – An excavation or opening, either controlled or a result of a failure of the dam, through a dam or spillway that is capable of completely draining the reservoir down to the approximate original topography, so the dam will no longer impound water, or partially draining the reservoir to lower impounding capacity. An uncontrolled breach is 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.

Chimney drain – A vertical or inclined layer of permeable material in an embankment to control drainage of the embankment fill.

Cofferdam – A temporary structure that encloses all or part of the construction area so that work can proceed in dry conditions. A diversion cofferdam diverts a stream into a pipe, channel, tunnel, or another watercourse.

Compaction – Mechanical action that increases soil density by reducing voids.

Concrete lift – The vertical distance between successive horizontal construction joints

Conduit – A closed channel to convey water through, around, or under a dam.

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 dissimilar materials, i.e., between a concrete tunnel lining and the surrounding rock.

Core wall – A wall built of impervious material, usually of concrete or asphaltic concrete in the body of an embankment dam to prevent seepage.

Creep – A process of deformation that occurs in many materials where the load is applied over an extended period.

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 next to the dam.

Dam – Any artificial barrier including appurtenant works constructed across rivers or tributaries thereof with a view to impound or divert water; includes barrage, weir and similar water impounding structures but does not include water conveyance structures such as canal, aqueduct and navigation channel and flow regulation structures such as flood embankment, dike and guide bund.

Dam failure – Failures in the structures or operation of a dam which may lead to an uncontrolled release of impounded water resulting in downstream flooding affecting the life and property of the people.

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 inspection of non-overflow section, 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 – The practice 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 predict deficiencies and consequences related to the failure and to document, publicize, and reduce, eliminate, or remediate to the extent possible, any unacceptable risks.

Densification – A means of improving the strength of soil by making it denser, usually by physical compaction.

Design and Construct – A form of contract in which the contractor undertakes both the design and the construction of the work.

Maximum water level – The highest water elevation, including the flood surcharge, that a dam is designed to withstand.

Design wind – The most severe wind that is possible at a reservoir for generating wind set-up and run-up. The determination will include the results of meteorological studies that combine wind velocity, duration, direction and seasonal distribution characteristics in a realistic manner.

Diaphragm wall – A cutoff wall of flexible concrete constructed in a trench cut through an embankment or the foundation.

Diversion dam – A dam built to divert water from a waterway or stream into a different watercourse.

Earth-fill dam – An embankment dam in which more than 50% of the total volume is formed of compacted earth layers.

Effective crest of the dam – The elevation of the lowest point on the crest (top) of the dam, excluding spillways.

Embankment dam – Any dam constructed of excavated natural materials, such as both earth-fill and rock-fill dams, or of industrial waste materials, such as a tailings dam.

Embankment zone – An area or part of an embankment dam constructed using similar materials and similar construction and compaction methods throughout.

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.

Emergency spillway – An auxiliary spillway designed to pass a large, but infrequent, volume of flood flow, with a crest elevation higher than the principal spillway or normal operating level.

Extensometer – An instrument used to detect, usually small, movements of a structure or a mass of rock or soil.

Failure mode – 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 can lead to an uncontrolled release of the reservoir.

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 – 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.

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.

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 case of a flood.

Flip bucket – An energy dissipater found 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 hydrograph – A graph showing, for a given point on a stream, the discharge, height, or another characteristic of a flood with respect to time.

Freeboard – Vertical distance between a specified reservoir surface elevation and the top of the dam, without camber.

Gabion – Rectangular-shaped baskets or mattresses fabricated from wire mesh, filled with rock, and assembled to form overflow weirs, hydraulic drops, and overtopping protection for small embankment dams. Gabion baskets are stacked in a stair-stepped fashion, while mattresses are placed parallel to a slope. Gabions have advantages over loose riprap because of their modularity and rock confinement properties, thus giving erosion protection with less rock and with smaller rock sizes than loose riprap.

Gallery – A passageway in the body of a dam used for inspection, foundation grouting, and/or drainage.

Gate – A movable water barrier for the control of water.

Gravity dam – A dam constructed of concrete and/or masonry that relies on its weight and internal strength for stability.

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 lines, in the foundation along which grout is injected to reduce seepage under or around a dam.

Hazard potential – The possible adverse incremental consequences that result from the release of water or stored contents because of failure or incorrect operation 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 measure of the potential for loss of life, property damage, or economic impact in the area downstream of the dam in case of a failure or malfunction of the dam or appurtenant structures. The hazard classification does not represent the physical condition of the dam.

Height of Embankment dam – The difference in elevation between the natural bed of the watercourse or the lowest point on the downstream toe of the dam, whichever is lower, and the effective crest of the dam.

Height of Concrete/Masonry dam – The difference in elecation between the lowest foundation level and the dam top elevation

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.

Hydrometeorology – The study of the atmospheric and land-surface phases of the hydrologic cycle with emphasis on the inter-relationships involved.

Hydrostatic pressure – The pressure exerted by water at rest.

Inclinometer – An instrument, usually consisting of a metal or plastic casing inserted in a drill hole and a sensitive monitor either lowered into the casing or fixed within the casing. The inclinometer measures the casing's inclination to the vertical at different points. The system may be used to measure settlement.

Inflow design flood –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, the height of the dam, and freeboard requirements.

Instrumentation – An arrangement of devices installed into or near dams that enable measurements that can be used to evaluate the structural behavior and performance parameters of the structure.

Internal erosion – 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 because of a dam failure. A series of maps for a dam could show the incremental areas flooded by larger flood releases. For breach analyzes, this map should also show the time to flood arrival, and maximum water-surface elevations and flow rates.

Jet grouting – A system of grouting in which the existing foundation material is mixed in situ with cementitious materials to stabilize the foundation, or it improve its water-tightness.

Karstic – An adjective to describe a limestone rock mass in which large openings have been caused over geological time by ground water dissolving the rock.

Large dam – 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.

Liquefaction – A condition whereby soil undergoes continued deformation at a constant low residual stress or with low residual resistance, because of the buildup and maintenance of high pore-water pressures, which reduces the effective confining pressure to a very low value. Pore pressure buildup leading to liquefaction may be due either to static or cyclic stress applications, and the possibility of its occurrence will depend on the void ratio or relative density of a cohesionless soil and the confining pressure.

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 used for emptying or for scouring sediment and sometimes for irrigation releases.

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.

Masonry dam – Any dam constructed mainly of stones, with cement mortar. in between.

Maximum storage capacity – The volume, in millions of cubic meters (Mm^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 because of sedimentation, or increase if the reservoir is dredged.

Normal storage capacity – The volume, in millions of cubic meters (Mm^3), 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.

Outlet – A conduit or pipe controlled by a gate or valve, or a siphon, that is used to release impounded water from the reservoir.

Outlet gate – A gate controlling the flow of water through a reservoir outlet.

Outlet works – A dam appurtenance that provides release of water (generally controlled) from a reservoir.

Parapet wall – A solid wall built along the top of a dam (upstream or downstream edge) used for ornamentation, for the 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.

Phreatic surface – The free surface of water seeping at atmospheric pressure through soil or rock.

Piezometer – An instrument used to measure water levels or pore water pressures in embankments, foundations, abutments, soil, rock, or concrete.

Piping – The progressive development of internal erosion by seepage.

Plunge pool – A natural or artificially created pool that dissipates the energy of free falling water.

Post-tensioned anchors – A system of anchored stressed steel tendons or bars within or attached to a structure to provide structural support.

Pre-stressed structure – A structure containing elements that have been pre-loaded with stressed steel tendons, bars or jacks.

Pressure relief pipes – Pipes used to relieve uplift or pore water pressure in a dam foundation or in the dam structure.

Probable Maximum Flood – The flood that may be expected from the most severe combination of critical meteorological and hydrologic conditions that are possible in the drainage basin under study.

Probable Maximum Precipitation – 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.

Principal spillway – The primary or initial spillway engaged during a rainfall-runoff event that is designed to pass normal flows.

Proposed dam – Any dam not yet under construction.

Radial gate – A gate with a curved upstream plate and radial arms hinged to piers or other supporting structure. Also known as a Tainter gate.

Rehabilitation – 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.

Repairs – Any work done on a dam that may affect the integrity, safety, and operation of the dam.

Reservoir – Any water spread that contains impounded water.

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. Specific types of storage in reservoirs are defined as follows:

a) 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.

b) Dead storage – The storage that lies below the invert of the lowest outlet and that, therefore, cannot readily be withdrawn from the reservoir.

c) Flood surcharge – The storage volume between the top of the active storage and the design water level.

d) Inactive storage – The storage volume of a reservoir between the crest of the invert of the lowest outlet and the minimum operating level.

e) Live storage – The sum of the active and the inactive storage.

f) Reservoir capacity – The sum of the dead and live storage of the reservoir.

g) Surcharge – The volume or space in a reservoir between the controlled retention water level and the highest 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.

Riprap – A layer of large rock, precast blocks, bags of cement, or other suitable material, placed on an embankment or along a watercourse as protection against wave action, erosion, or scour.

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. The risk analysis should include all potential events that would cause an unintentional release of stored water from the reservoir.

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.

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 bolt – A tensioned reinforcement element consisting of a steel rod, a mechanical or grouted anchorage, and a plate and nut for tensioning or for retaining tension applied by direct pull or by torquing.

Rock reinforcement – The placement of rock bolts, un-tensioned rock dowels, pre-stressed rock anchors, or wire tendons in a rock mass to reinforce and mobilize the rock's natural competency to support itself.

Rockfill dam – An embankment dam in which more than 50% of the total volume is composed of compacted or dumped cobbles, boulders, rock fragments, or quarried rock larger than 75-millimeter size.

Roller compacted concrete dam – A concrete gravity dam constructed using 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 not shaped or coursed.

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.

Seepage – The internal movement of water that may take place through a dam, the foundation or the abutments, often emerging at the ground level lower down the slope.

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.

Settlement – The vertical downward movement of a structure or its foundation.

Shotcrete – Concrete sprayed through a nozzle onto the surface to be covered.

Sinkhole – A depression that indicates sub-surface 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. Can be estimated from wind speed, fetch length, and wind duration

Slide – Movement of a mass of earth down a slope on the embankment or abutment of a dam.

Slide gate – A gate that can be opened or closed by sliding in supporting guides.

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.

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.

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 to provide a cheaper or more easily handled means of temporary closure than a bulkhead gate.

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 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 the ground surface is called the heel or the upstream toe.

Top thickness (top width) – The thickness or width of a dam at the level of the top of the dam (excluding corbels or parapets).

Trash rack – A device found at an intake to prevent floating or submerged debris from entering the intake.

Uplift – The hydrostatic force of water exerted on or underneath a structure, tending to cause a displacement of the structure.

Volume of dam – The total space occupied by the materials forming the dam structure computed between abutments and from top to bottom of the dam. No deduction is made for small openings such as galleries, adits, tunnels, and operating chambers within the dam structure. The volumes of power plants, locks, and spillways are included only if they are needed for structural stability of the dam.

Wave protection – Riprap, concrete, or other armoring on the upstream face of an embankment dam to protect against scouring or erosion caused by 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 barrier across a stream designed to alter its flow characteristics. In most cases, weirs take the form of obstructions smaller than conventional dams, pooling water behind them while also allowing it to flow steadily over their tops.

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 consists of a rectangular, trapezoidal, triangular, or another 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.

Wind setup – The vertical rise in the stillwater level at the face of a structure or embankment caused by the wind stresses acting on the surface of the water.

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DR. ANIL K CHOPRA

Anil K. Chopra received his Bachelor of Science degree in Civil Engineering from Banaras Hindu University, India, in 1960, the Master of Science degree from the University of California, Berkeley, in 1963, and the Doctor of Philosophy degree, also from Berkeley, in 1966. He joined the faculty at the University of California, Berkeley, in 1969 where he has served as Vice Chair (1980-83) and Chair (1991-93, 1994-97) of the Structural Engineering, Mechanics and Materials program in the Department of Civil and Environmental Engineering. He has been responsible for the development and teaching of courses in structural engineering, structural dynamics, and earthquake engineering. His research activities have included studies of structural dynamics, various problems in earthquake analysis and design of buildings, dynamic soil-structure interaction, dynamic fluid-structure interaction, and earthquake analysis and design of concrete dams. He has authored more than 380 published papers on this work, a monograph, *Earthquake Dynamics of Structures, A Primer*, 2005, and a textbook, *Dynamics of Structures: Theory and Applications to Earthquake Engineering*, 1995, 2001, 2007, 2012, and 2017. Professor Chopra has served as a consultant on earthquake engineering problems to numerous governmental and private organizations in several countries. Currently, he serves on the Consulting Board for Earthquake Analysis, Division of Safety of Dams, Department of Water Resources, State of California. The analytical methods and computer programs (EAGD-84, EACD-3D etc.) developed under his supervision for the earthquake analysis of concrete dams have been adopted as standards by agencies such as the U.S. Bureau of Reclamation and the U.S. Army Corps of Engineers, and have been utilized by many organizations in the design of new dams and in the seismic evaluation of existing dams.

LARRY KENT NUSS, P.E.

Larry retired from the Bureau of Reclamation on December 31, 2011 with 36 years of experience in the design, structural analyses (static, thermal, and seismic), dam safety, risk analysis, and security of concrete dams (gravity, arch, spillway, and buttress dams). He formed Nuss Engineering, LLC on January 20, 2012. Duties have included advanced dynamic structural analysis, Team Leader on Issue Evaluations and Corrective Action Studies, Technical Approval, Peer Review, mentoring, site inspection; Comprehensive Facility Review, risk analyses facilitator and member, and member of dam safety and security advisory teams. He has been a Consultant Review Board Member on projects in Panama, Australia, New Zealand, Turkey, Canada, and over 20 projects in the USA, namely with the Tennessee Valley Authority, Bureau of Reclamation, Corps of Engineers, New Brunswick Power, Hydro Quebec, Federal Energy Regulatory Commission, Xcel Energy, FortisBC, Southern California Edison, Panama Canal Authority, Trust Power (New Zealand), AGL Hydro (Australia), EnergiSA (Turkey), HDR, Stantec/MWH, Hatch, AECOM, Shannon & Wilson, Kleinfelder, Intertechne (Brazil), Hatch, Schnabel, DLZ, and Tonkin & Taylor (Australia). He has taught classes in the United States, Japan, China, and Brazil. He has a Master of Science, University of Colorado, Boulder, Colorado, in Civil Engineering in 1978. He is currently licensed as a Professional Engineer in the State of Colorado, No. 17615 and a Practicing Professional Engineer, New Brunswick, Canada, No. L4915.

Central Dam Safety Organisation

Central Water Commission

Vision

To remain as a premier organisation with best technical and managerial expertise for providing advisory services on matters relating to dam safety.

Mission

To provide expert services to State Dam Safety Organisations, dam owners, dam operating agencies and others concerned for ensuring safe functioning of dams with a view to protect human life, property and the environment.

Values

Integrity: Act with integrity and honesty in all our actions and practices.

Commitment: Ensure good working conditions for employees and encourage professional excellence.

Transparency: Ensure clear, accurate and complete information in communications with stakeholders and take all decisions openly based on reliable information.

Quality of service: Provide state-of-the-art technical and managerial services within agreed time frame.

Striving towards excellence: Promote continual improvement as an integral part of our working and strive towards excellence in all our endeavours.

Quality Policy

We provide technical and managerial assistance to dam owners and State Dam Safety Organizations for proper surveillance, inspection, operation and maintenance of all dams and appurtenant works in India to ensure safe functioning of dams and protecting human life, property and the environment.

We develop and nurture competent manpower and equip ourselves with state of the art technical infrastructure to provide expert services to all stakeholders.

We continually improve our systems, processes and services to ensure satisfaction of our customers.

Dam Rehabilitation & Improvement Project