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**SCHOOL OF GRADUATE STUDIES
SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING**

**Comparative Performance Evaluation of Roller- Compacted Concrete and
Conventional Vibratory Concrete Gravity Dams Under Earthquake Loads:
A Case Study of Gibe III Dam in Ethiopia**

A Thesis Submitted to the School of Graduate Studies of Civil and Environmental
Engineering of Addis Ababa Institute of Technology in Partial Fulfillment of the
Requirements for the Degree of Master of Science in Civil Engineering
(Major in Dam Engineering)

By: Alemayehu Mamo Mellese

Advisor: Dr.-Ing.Asie Kemal Jabir

**November 2023
Addis Ababa, Ethiopia**

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SCHOOL OF GRADUATE STUDIES
ADDIS ABABA INSTITUTE OF TECHNOLOGY

APPROVAL OF THE THESIS

This is to certify that the thesis prepared by **Alemayehu Mamo Mellese**, entitled “**Comparative Performance Evaluation of Roller-Compacted Concrete and Conventional Vibratory Concrete Dams under Earthquake Loads: A Case Study of Gibe III Dam in Ethiopia**,” submitted to the School of Graduate Studies of Addis Ababa University in Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering (Major in Dam Engineering), complies with the regulations of the university and meets the accepted standards with respect to originality and quality.

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CERTIFICATION

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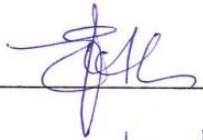
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I, **Alemayehu Mamo Mellese**, hereby declare that this thesis is my own original work and it has not been presented and will not be presented by me to any other university for similar or any other degree award. All information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

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ABSTRACT

Ethiopia is currently planning to build many large, medium and small dams as a step forward to the development of the hydropower and irrigation potentials in the different regions of the country. Rolled Compacted Concrete (RCC) dams are among the technologies that have been recently introduced for hydroelectric projects. The recently completed Gibe III dam, a 243m high dam, located on the Omo river in the south-western part of Ethiopia, is located in the vicinity of the seismically active region of the main East African Rift System (EARS). Many existing concrete dams, including Gibe III dam, have been in operation for decades in several countries, however their capacity to perform well during major seismic events is yet to be confirmed when seismic performance criteria have changed and/or new developments have taken place in seismic hazard assessment, in defining design/evaluation earthquakes, in methods of seismic analysis, and in the dynamic behavior of materials. In this study, Gibe III RCC dam was selected as a case study and its seismic performance was compared with an idealized equivalent CVC dam. Project information for the existing dam and previous site-specific seismic studies were reviewed; design/evaluation earthquakes, as recommended by the ICOLD (2010), were defined; ground motion acceleration-time histories were selected from PEER-NGA database, and then modified (scaled) using SeismoMatch 2022. The comparative seismic performance evaluation was carried out using linear-elastic time history analysis procedures with the help of finite element simulation program, ANSYS Workbench 2023 (Transient Structural). The seismic responses (deformations and stresses) were evaluated and compared in accordance with the acceptance criteria provided in the US Army Corps of Engineers (USACE, 2007) guidelines. Based on the analyses results, it was found that both types of dams have demonstrated high degree of similarities in deformations and stresses when subjected to critical earthquake load combination cases comprising Operating Basis Earthquake (OBE) and Safety Evaluation Earthquake (SEE). In terms of deformation behaviors, however, the results for the CVC dam were found slightly lower in magnitude than that for the RCC dam type, which resulted from the use of a homogeneous and higher grade of mass concrete (18MPa compressive strength) in the CVC dam as opposed to the use of variable compressive strengths (8 to 18MPa) in different regions of the RCC dam. The study also concluded that the tensile stress responses of both types of dams to OBE ground motions in reservoir empty condition are falling inside the performance limit, indicating “little or no damage.” However, their responses to OBE and SEE ground motions in reservoir full conditions are found to exhibit nonlinear responses, indicating the need for nonlinear analysis using 2-D and/or 3-D finite element modelling. In the Ethiopian context, considering the potential advantages that are possibly gained in large proportions (time, economy, methodology) from the RCC method of construction, the choice of RCC method of construction is highly recommended as this method allows the use of wide range of material properties and variable strengths of concrete in different regions that are sometimes unacceptable or unsuitable in the case of CVC method.

Keywords: gravity dams, seismic design, seismic performance evaluation, roller-compacted concrete, conventional mass concrete, dynamic analysis, time-history analysis, operating basis earthquake, safety evaluation earthquake, PEER-NGA.

DEDICATION

I dedicate this thesis to the memory of my late father, Sergeant Mamo Mellese, who truly played the greatest and unmatched roles in my life.

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ACRONYMS AND ABBREVIATIONS

ACI	:	American Concrete Institute
ANAYS	:	Analysis Systems
CMC	:	Conventional Mass Concrete
CVC	:	Conventional Vibratory Mass Concrete
CES	:	Compulsory Ethiopian Standard
COSMOS	:	Consortium of Organizations for Strong- Motion Observation Systems
DCR	:	Demand-to-Capacity Ratio
DOF	:	Degree of freedom
DSHA	:	Deterministic Seismic Hazard Analysis
d/s	:	Downstream
EARS	:	East African Rift System
EEP	:	Ethiopian Electric Power
ENTRO	:	Eastern Nile Technical Regional Office
FEMA	:	Federal Emergency Management Agency
FEM	:	Finite Element Method
FEA	:	Finite Element Analysis
GE-RCC	:	Grout Enriched Roller-Compacted Concrete
GERD	:	Grand Ethiopian Renaissance Dam
GOE	:	Government of Ethiopia
GPa	:	Giga Pascal
GSI	:	Geological Strength Index
ICOLD	:	International Commission on Large Dams
kJ	:	Kilo Joule
MCE	:	Maximum Credible Earthquake
MDE	:	Maximum Design Earthquake
MDOF	:	Multiple Degree of Freedom
MPa	:	Mega Pascal
M _w	:	Moment magnitude
M _s	:	Surface wave magnitude
MW	:	Megawatt
NGA-West2	:	Next Generation Attenuation Relationships for Western United States
NMSA	:	Nominal Maximum Size of Aggregates
OBE	:	Operating Basis Earthquake
PGA	:	Peak Ground Acceleration
PSHA	:	Probabilistic Seismic Hazard Analysis
PEER	:	Pacific Earthquake Engineering Research Center
PEER-NGA	:	Pacific Earthquake Engineering Research-New Generation Attenuation
PGMD	:	Peak Ground Motion Database
PGA	:	Peak Ground Acceleration

PGHA	:	Peak Ground Horizontal Acceleration
PMF	:	Probable Maximum Flood
RP	:	Return period
RCC	:	Roller-Compacted Concrete
RTS	:	Reservoir Triggered Seismicity
SANCOLD	:	South African National Committee on Large Dams
SEE	:	Safety Evaluation Earthquake
SPF	:	Standard Project Flood
SDOF	:	Single Degree of Freedom
USBR	:	United States Bureau of Reclamation
USCOLD	:	United States Committee on Large Dams
USSD	:	United States Society on Dams
USACE	:	United States Army Corps of Engineers
u/s	:	Upstream
WES	:	Waterways Experiment Station
2-D	:	Two-dimensional
3-D	:	Three-dimensional

SYMBOLS

$V_{s,30}$	:	Average shear wave velocity in top 30m depth
g	:	Acceleration due to gravitational force (9.81m/s ²).
a_g	:	Seismic horizontal ground acceleration
a_{vg}	:	Seismic vertical ground acceleration
C'	:	Cohesion
σ	:	Normal stress
Φ'	:	Angle of internal friction
f_c'	:	Characterstic compressive strength of concrete
f_t'	:	Direct tensile strength of concrete
f_{st}'	:	Split tensile strength of concrete
E_c	:	Young's modulus of concrete
E_w	:	Young's modulus of water
E_d	:	Dynamic modulus of elasticity of concrete
E_f	:	Young's modulus of elasticity of foundation
E_m	:	Deformation modulus of foundation (rockmass)
ν	:	Poisson's ratio
τ	:	Shear stress
W	:	Watt
K	:	Bulk modulus
S_a	:	Spectral Acceleration
$V_s.$	:	Versus

1 INTRODUCTION

1.1 Background

Dams were likely among the earliest large constructions that mankind had created and built. In many countries around the world, dams have a substantial positive impact on the social and economic wellbeing of the general populace as well as the growth of the nation's overall infrastructure. Over the past 80 years, dam engineering has undergone a significant development. During this time, they have been employed to meet a variety of essential human needs, including water supply, irrigation, power generation, recreational, and navigational functions.

As early as the 1930s, large dams were among the first constructions to be studied for seismic design requirements. Until 1989, when the International Commission on Large Dams (ICOLD, 1989) published the recommendation on "Selecting Seismic Parameters for Large Dams" (Bulletin 72), it was normal practice to build dams against earthquakes using the pseudo-static approach, usually with a horizontal acceleration of $0.10g$. Nevertheless, the maximum ground shaking at a dam site generated by a powerful shallow focus earthquake can exceed the original pseudo-static design acceleration of $0.10g$.

Earthquake safety requires careful consideration. The fundamental idea of seismic safety—that a dam with a high risk of failure must be able to withstand the strongest ground motion anticipated at the dam site—remains in place. Numerous earthquake occurrences have demonstrated that dams continue to be seriously at risk from the earthquake danger. Additionally, the study of seismic hazard analysis has advanced significantly over the past years, and estimates of seismic hazard have been continually increasing. Additionally, albeit much more slowly than the seismic hazard analysis methods, seismic design and performance criteria and procedures have also advanced. Majority of the existing dams constructed before the 1990s were designed against earthquakes using seismic design criteria and/or methods of dynamic analysis, which are regarded outdated or even incorrect today. Therefore, the seismic safety of these dams is unknown if contemporary criteria are used, and it is necessary to suppose that some of them have structural flaws. As a result, it is considered mandatory that both major and minor dams' earthquake safety has to be systematically reevaluated (Wieland 2003, 2006).

1.2 Overview of the Need for Seismic Performance Reassessment

Earthquakes have historically caused major damage to few concrete dams. Only approximately a dozen dams have fully failed as a result of an earthquake worldwide. The majority of these dams were either tailings or hydraulic fill dams, or they were relatively aged, earthfill embankments of possibly poor design. A number of additional large embankments or concrete gravity dams have been seriously destroyed. Several of the embankment dams failed almost completely and had to be reconstructed.

Generally, Overall, the seismic performance of concrete dams has been satisfactory, and they are therefore practically found to be more earthquake-resistant than embankment dams. This could be because concrete dams were built to higher design standards than some of the older embankment dams.

Additionally, concrete dams are presumably less prone to aging, material deterioration, seepage, and inadequate maintenance than earlier embankment dams. However, with the exception of a few dam sites, many existing concrete dams in seismically active areas have not been tested by levels of ground motion equivalent to the applicable design earthquake, and hence the actual test of performance has yet to occur (USSD, 2014). The USSD's study (2017) suggested that the great performance records may be largely due to the fact that few dams have been shaken by earthquakes of sufficient duration and severity to compromise their structural integrity. To this end, the comprehensive potential seismic failure mode for concrete dams has not been fully realized or experienced, and the actual reaction of enormous concrete structures to earthquake ground motions is extremely complex (USACE, 2007).

While concrete dams are built to endure higher levels of seismic ground shaking and have done well in the past, we should not be overly optimistic about their future performance. The fact that no large dams have failed or collapsed during earthquakes and just a few people have lost their lives may convey the impression that well-designed dams are safe against earthquakes. We need to re-evaluate the seismic safety of existing dams based on current best practices and, if necessary, renovate existing dams. The design details, updated seismic design and evaluation criteria, and construction quality should all be carefully considered. To accomplish a satisfactory evaluation of dam earthquake safety, meaningful or justifiable seismic parameters must be used. Proper seismic evaluation will not replace, but will supplement, the use of sound design, high-quality materials, good construction control processes, and constant surveillance and monitoring of the final structure's performance. It is important to note that, regardless of the seismic parameters and analysis methods chosen, the final assessment of the dam's seismic safety typically depends to some extent on engineering judgment and prior experience with similar structures in connection with evaluating newly developed research methods and newly obtained research results, with the fact that each completed dam structure and its environment represent a unique system that does not exist elsewhere in a different geographic location (Wieland, 2006).

Having noted the above explanations, the need for periodic seismic performance evaluation of concrete gravity dams is justified from this perspective.

1.3 Statement of the Problem

In recent years, several regions of the world have begun to pay more attention to the seismic safety of dams. It is now taken into account heavily when planning new dams that are envisioned being built in seismically active regions as well as when evaluating the safety of existing dams in those locations.

Since the 1970s, conventional vibratory concrete (CVC) and roller-compacted concrete (RCC) dams have been the most recent innovations in dam construction techniques. Due to the significant potential advantages of RCC dams, they have emerged as viable alternatives to large-sized conventional concrete structures. Case studies on the seismic performance of numerous concrete dams worldwide revealed that RCC dams are anticipated to function quite similarly to CVC dams under seismic loads (Wieland et al., 2004; USSD, 2017). The USSD's study (2017) also concludes that the entire possible seismic failure mechanisms (cracks, deformations, sliding, joint displacement, openings, shearing, or other failure modes) for concrete dams has not been fully realized or experienced. It has also been discovered that the real response of large concrete dam structures to earthquake ground motions is quite complex (USACE, 2007). It is important to note that the dynamic response of RCC and CVC dams to earthquakes requires further research using appropriate modeling techniques and analysis methods. Thorough research and examinations are required to determine the seismic performance of RCC and CVC dams under strong ground shaking as well as any potential issues and weaknesses of each type of dam as the design and construction practices are still in development (Wieland et al., 2004).

Therefore, understanding how each method of construction – RCC vs. CVC - responds to severe loading conditions under earthquake loads is of paramount importance in the feasibility study, planning and implementation of new dams and in the safety evaluations of existing dams like Gilgel Gibe III dam built in seismically active regions and other potential dam sites in East African Rift System (EARS). Additionally, it is recommended by many researchers that several seismic analyses be carried out throughout a life span of the dam during which seismic design criteria or seismic performance criteria have changed and/or new advancements have been made in the assessment of seismic hazard, seismic analysis techniques, or dynamic behavior of materials, etc. (Wieland, 2006). As a result, it is necessary to periodically review or re-evaluate the seismic safety of existing dams using current best practices and the most recent seismic safety and performance standards. This helps in the planning of further rehabilitation procedures for existing major concrete dams like Gibe III dam in the event that any uncertainties arise in the life time of the dam. Therefore, the need for periodic reviews or re-evaluation of the seismic safety of existing dams and comparison between equivalent RCC and CVC dams based on the current state-of-the-art practice using up-to-date seismic safety and performance criteria is justifiable considering Gibe III dam as a case study of the seismic performance evaluation in a seismically active regions within the Main Ethiopian Rift System. The study also helps

in the planning for further rehabilitation measures, as necessary, in the event that any up-to-date seismic evaluation criteria are not satisfied by the case study.

1.4 Research Questions

Several researches indicated that the conventional method of designing and evaluating concrete gravity dams under earthquakes – static or dynamic analysis approaches – have not been found adequate or comprehensive to illustrate the entire potential seismic failure modes for concrete dams. Due to the very complex nature of earthquake analysis in determining actual response of massive concrete structures to earthquake ground motions, the existing knowledge gaps and shortfalls need to be identified at various stages to ensure that a more realistic representation of the challenges and solution process is achieved. More specifically, the following research questions are needed to be addressed under this research paper:

- (1) What are the presently accepted state-of-the-art methods, guidelines, and practices being applied for the seismic analysis and performance evaluation of concrete gravity dams?
- (2) What are the similarities and differences in the seismic response behaviors (specifically deformations and stresses) of RCC and CVC dams? What are the major factors or parameters contributing to the similarities and differences in seismic responses of deformations and stresses?
- (3) Which type of dam construction method is most suitable in terms of earthquake resistance and other design parameters in the Ethiopian context? What are the specific contributions of this research findings and recommendations in the context of Ethiopia considering the national development plans for building many more dams of similar types for different purposes?

1.5 Objectives of the Study

Ethiopia is planning to build many more dams as part of the development and transformation plans and most of the potential dam sites are located in seismic regions in which defining the seismic design criteria, analysis method and performance evaluation will be the primary concern of dam owners and the basis on which the long-term existence/stability and serviceability of the dams will be justified in compliance with established performance criteria of recognized standards/guidance.

This thesis presents the dynamic response analysis and comparative seismic performance of roller-compacted concrete and conventional vibratory concrete dams by considering different critical load combinations. The study was made for the case of Gibe III dam which is regarded as the tallest and largest RCC dam that was recently built and currently in service in Ethiopia. The seismic performance evaluation of the two types of dams was carried out in accordance with the methodology and guidelines outlined in the United States Army Corps of Engineers (USACE, 2007) publications.

1.5.1 General Objectives

The general objectives of the study is to analyze and compare the seismic performance of RCC and CVC gravity dams by considering the case of Gibe III dam using state of the art practices, recent developments of methods of dynamic analysis, seismic hazard assessment, seismic design and performance criteria.

1.5.2 Specific Objectives

The specific objectives of the study include the following:

- (1) To provide general framework and guidelines for understanding earthquake design features, considerations, and solution process for seismic performance evaluation of RCC and CVC gravity dams;
- (2) To perform a two-dimensional linear-elastic-time history analysis of the dam–reservoir–foundation system for the existing Gibe III RCC dam and an equivalent idealized CVC gravity dam and determine their dynamic response histories (time-varying displacements, strains, stresses) under the combined static loads and horizontal components of earthquake ground motions using Finite Element SiMethod with the help of a finite element simulation and analysis program called ANSYS Workbench 2023 (Transient Structural component).
- (3) To compare their dynamic response behaviors – stress and deformation response histories – when subjected to combined static loads and horizontal components of earthquake ground motions (OBE and SEE), and put forward recommendations following a rational methodology and prescribed internationally recognized performance evaluation procedures and criteria (USACE 2007 guidelines).

By doing so, the analysis results, conclusions and recommendations drawn from the above exercises will be useful in bringing the earthquake problems of concrete dams to the attention of government agencies and/or dam owners that may initiate, direct, or fund research and to provide them with information helpful in the planning, implementation and operation of similar dam structures in the Ethiopian context.

The seismic performance evaluation was limited to the above-mentioned two most common types of concrete gravity dams. For the purpose of comparative seismic response analysis and performance evaluation, the existing Gibe III RCC dam was compared with an idealized CVC dam with similar cross-sectional dimensions and recommended material properties for static and dynamic properties as prescribed by USACE (2007) and related guidelines.

1.6 Significance of the Study

The main reason for conducting this study is that Ethiopia is currently constructing several dams among which concrete gravity dams are the common choice for large dams for multipurpose uses. Many design reports for several dams show there is high degree of variability in the assumptions, design criteria and methodology of analysis. For some dams, the current state-of-the-art and recent developments are adopted, while for others, outdated codes and guidelines are being adopted. Therefore, the seismic performance of these dams is not well understood as the general trend is a subjective approach rather than being a consistent methodology for estimating ground motion parameters that conforms to acceptable national or international standards and guidelines. Hence review of their seismic performance as recommended by ICOLD (2010) is of paramount importance in building up the confidence levels on the sustainability of the dams.

The guidelines will provide a basic framework for the earthquake design and evaluation of dams, as well as for the planning, design, construction, operation, maintenance, and regulation of dams. The general guidelines will help in specifying design earthquake loadings (for design or safety evaluation); performing seismic analyses for the design of new dams (for evaluating the safety of existing dams or modifying existing dams); and establishing definite recommendations as to the appropriate type of dam construction method – RCC or CVC type - that will perform better with adequate safety margin in seismically active regions.

Furthermore, it is also recommended that, during the lifespan of a dam, several seismic analyses are needed to be performed when the seismic design criteria or seismic performance criteria have changed and/or new developments have taken place in the seismic hazard assessment, in the methods of seismic analysis, or in the dynamic behavior of materials.

The study therefore provides dam owners and associated stakeholders with recommendations for the appropriate choice of dam types in response to severe load combinations with seismic loads and apply defensive design measures when designing new dams of similar types. The defensive design measures for concrete gravity dams include: sufficient foundation and abutment exploration, material testing, strengthening; adequate drainage system; use of best geometric design and structural detailing; effective quality control during construction; etc.

1.7 Scope and Organization of the Study

This thesis is organized into five (5) chapters as briefly explained below:

Chapter 1 presents a brief introduction to the overall background information about the seismic performance of concrete gravity dams and the importance of periodically performing seismic performance evaluation of existing concrete dams using state-of-the art practices and updated seismic design guidelines and researches. In addition, the need for the research and the significance of the study are discussed. .

Chapter 2 presents detailed literature reviews conducted for the research focusing on the basic engineering concepts and developments, assumptions, design considerations, methods of analysis, procedures, and sound engineering practices and standards being applied in the design and performance evaluation of concrete gravity dams with more emphasis on roller-compacted concrete and conventional concrete dams.

Chapter 3 presents the methods and materials used in conducting this study. It describes the brief description of the study area, the materials and data collection, development of structural analysis modelling, analysis methodology and computer software used for the linear-elastic time history analysis, seismic performance evaluation and comparison of results for RCC and CC dams. It also provides the procedures adopted for the dynamic response analysis by considering Gibe III dam as the case study in the context of Ethiopia.

Chapter 4 presents the detailed dynamic analysis results, discussions, and validation of methodology followed in this study.

Chapter 5 summarizes the researcher's conclusions and recommendations drawn from this study.

Appendices: This research contains two appendices:

Appendix I contains the original (unscaled) acceleration-time histories [RSN454_MORGAN_GIL337.AT2] obtained from PEER ground motion database and used in the time history analysis using OBE loading case in this study.

Appendix II contains the original (unscaled) acceleration-time histories [RSN1633_MANJIL_ABBAR--L.AT2] obtained from PEER ground motion database and used in the time history analysis using SEE loading case in this study.

2 LITERATURE REVIEW

This section deals with the literature reviews focused on the basic design concepts, assumptions, design considerations, methods of analysis, procedures, and sound engineering practices and standards being applied in the design and performance evaluation of concrete gravity dams with more emphasis on roller-compacted concrete and conventional concrete dams. This serves as a guide to sound engineering practices to become aware of the problems that may require special attention during the planning, design, operation and maintenance of similar types of concrete gravity dams in a highly seismic regions.

2.1 Classification of Dam Types

According to USBR (1987) dams may be classified into a number of different categories based on different criteria. There are three broad categories: according to their use, their hydraulic design, or the materials of which they are constructed, as depicted in Figure 2-1 below.

The most typical classification for design techniques is employed based on the materials used to construct the dam structure. This categorization typically also acknowledges the fundamental design, such as the "concrete gravity" dam or the "concrete arch" dam; and additionally, there exists other types of dams such as brick, masonry and timber.

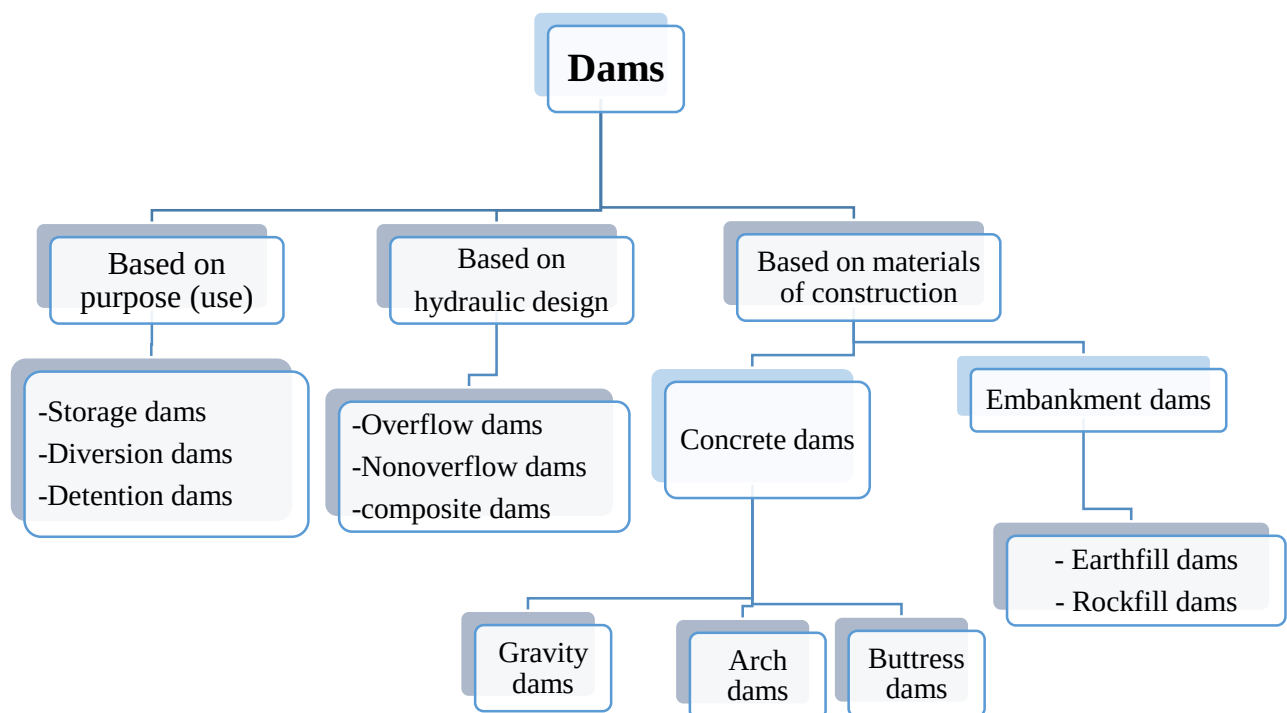


Figure 2-1 Classification of types of dams based on different criteria (USBR, 1987)

Concrete gravity dams can further be built using either conventional mass concrete (CMC) or roller-compacted concrete (RCC) methods, both of which are usually appropriate for locations with a moderately sound rock foundations. These types of dams are well suited for usage as overflow spillway crests. Gravity dams can have a curved or straight plan, and because of this benefit, they are frequently employed as spillways for earthfill or rockfill dams or as overflow sections of diversion dams.

2.2 Evolution of Modern Concrete Dams

According to the "Concrete Dam Evolution" study by USBR (2002), the lengthy history of world-renowned concrete dams began immediately after the turn of the twentieth century with the construction of landmark masonry dams. Throughout the 1920s, arch, gravity, and buttress dam architecture progressed. Significant advances in design, analysis, and construction were accomplished in the 1930s with the design and building of Hoover Dam. Concrete materials, temperature control, and construction procedures have also advanced. Concrete technology advanced in response to the challenges of alkali-aggregate reaction and freeze-thaw degradation caused by the Hoover Dam.

In the 1970s, the seismic response of concrete dams received major attention, and this incident was followed by the introduction of the finite element approach to these types of assessments. Roller-compacted concrete (RCC) was developed in the 1980s as a new form of concrete dam construction that utilised earth moving and paving technologies for transporting and placing concrete materials in bulk quantities, which resulted in faster construction periods and lower costs. Due to the advanced RCC materials design and placement methods, continued evaluations for dam safety, operations, and maintenance have been progressively advancing as the behavior and risks posed by these dams are better understood, modifications have been made for several concrete dams to improve their safety and service life.

Since the early days, especially between the years 1930 and 1970, enormous development has been made in mass concrete construction for dams, and the art and science of dam building practised today has reached a highly advanced state. RCC has now taken over as the primary mass concrete placement technique, but conventional mass concrete techniques are still employed for many projects, including both big and minor dams.

2.3 Concrete Gravity Dams

2.3.1 Definition

A concrete gravity dam is a solid concrete structure that is proportioned and designed in such a way that its weight is sufficient to provide majority of the resistance to all forces acting on it and assures stability against the effects of all imposed forces (USBR, 1976). Generally, they are constructed on a

straight axis, or may be slightly curved to conform to the specific site conditions. They typically consist of a nonoverflow section(s) and an overflow section or spillway (USACE,1995; USBR, 1987).

Concrete gravity dams are usually appropriate for places with a moderately solid rock foundation. Because they are well adapted for usage as overflow spillway crests, they are frequently employed as spillways for earthfill or rockfill dams or as overflow sections of diversion dams (USBR,1987). Concrete gravity dam will be a permanent structure that requires little maintenance if the foundation is suitable and the dam is appropriately designed and constructed

Based on the general construction methods, concrete gravity dams are classified into two types (USACE, EM 1110-2-2200, 1995):

- (1) Conventional Vibratory Mass Concrete (CVC) dams
- (2) Roller-Compacted Concrete (RCC) dams.

2.3.2 Conventional Vibratory Mass Concrete Dams

(a) Definition and Scope

According to ACI 116R, "mass concrete" is any volume of concrete whose dimensions are large enough such that special precautions must be taken to deal with the heat generated during the hydration of the cement and the resulting volume change to prevent cracking. Traditional mass concrete placement techniques and materials are used in the construction of mass concrete dams in order to properly proportion, mix, place, cure, and control temperature. Strength is frequently a secondary, rather than a major issue in mass concrete structure construction, with durability, economics, and thermal action being the main design considerations.

Conventional mass concrete gravity dams primarily rely on their geometric shape and weight to resist loads, and secondarily, through their strength.. Their properties in terms of durability and thermal behaviours and tendency for cracking are much more important in concrete gravity dams (ACI 207.1R-2, 2012).

Since the early days, notably between 1930 and 1970, significant progress has been made in mass concrete dam construction, and the art and science of dam building practiced today has grown to a highly advanced degree. The Hoover Dam, completed in 1935, signaled the start of an age of improved practices in large mass concrete dam construction. Although RCC was created and became the leading method for pouring mass concrete from 1970 to the present; nevertheless, conventional vibratory mass concrete (CVC) technologies continue to be utilized for many large and small dam projects, particularly where RCC would be impracticable or impossible to use (ACI 207.1R-4, 2012).

The advantage that has been foreseen by not using of the 28-day strength criterion as a design criteria for dams was an important development of the 1950s due to the reasons that maximum stresses under

loads do not normally arise until the concrete is at least a year old. Under mass curing circumstances, with the cement and pozzolans typically used, the gain in concrete strength, according to ACI 232.2R, might range from 30 to more than 100% between 28 days and 1 year. As a result, dam designers now practice defining the desired strength of mass concrete at later ages, such as 1 or 2 years.

(b) Construction and temperature control

Construction of CVC dams include batching and mixing, as well as transportation, placement, vibration, cooling, curing, and preparation of horizontal construction joints between lifts, which have been created and mastered over many years of designing and building mass concrete dams.

The concrete is poured in 1.5-3.0m depth lifts with each lift made up of layers that are no more than 45-50cm thick. The heat generated by cement hydration necessitates strict temperature control during mass concrete laying and for several days afterwards. Due to high gradients within the mass concrete or temperature drops as the concrete approaches its yearly temperature cycle, uncontrolled heat generation could result in excessive tensile stresses. Precooling and postcooling procedures are used as control measures to limit peak temperatures and control temperature decline. The potential for temperature rise has been decreased by lowering the cement content and replacing cement with pozzolans. The traditional concrete gravity dam is constructed in a series of individually stable monoliths, each monolith being typically around 15m wide, separated by transverse contraction joints to achieve crack control (USACE, EM 1110-2-2200, 1995).

To achieve crack control standards, the cement hydration process of conventional concrete limits the size and speed of concrete placing and mandates constructing in monoliths. Mix proportions are generally selected using large-size coarse aggregates to produce a low-slump concrete that provides economy, maintains good workability during placement, develops minimal temperature rise during hydration, and produces important properties such as strength, impermeability, and durability (USACE, EM 1110-2-2200, 1995).

2.3.3 Roller-Compacted Concrete Dams

(1) Definition, Scope and Development

Roller-compacted concrete (RCC) is defined by ACI 207.5R-11 (2011) as "concrete compacted by roller compaction; concrete that, in its unhardened state, will support a roller while being compacted." RCC is frequently mixed using high-capacity continuous mixing or batching equipment, transported using trucks or conveyors, and distributed in layers using bulldozers before being compacted using vibratory rollers. RCC doesn't resemble regular concrete visually until the cement has nearly reached its final setting point or until the hydrated interior is visible (Nawy, E.G., 2008).

Probably the most significant advancement in concrete dam technology in the last 25 years has been RCC. The work done with RCC in the 1970s served as the foundation for RCC dams, which first appeared in the 1980s and then had a significant rise in popularity. Due to the lower cost gained from the quick building approach, the usage of RCC has made several new dams economically feasible. Due to demand, economics, and RCC's outstanding performance, the technology has gained widespread acceptance quickly (ACI 207.5R-11, 2011; Nawy, E.G., 2008). During the 1970s, several organizations were involved in experiments, laboratory analyses, and the creation of various mass RCC ideologies.

Over the past several decades, although the United States initially had the largest number of RCC dams, they are now more common in nations like China, Spain, and Brazil, and their popularity is growing in Vietnam, India, and other Asian and Southeast Asian countries (Nawy, E.G., 2008).

(2) Construction

The most typical technique of spreading RCC is by dozer. RCC should typically be spread to the loose lift thickness required to provide a final lift thickness of 0.30m following compaction. It is critical that the RCC be delivered, deposited, disseminated, and compacted within 45 minutes of the mix water contacting the cementitious material (mixed with both cement and pozzolan) (USBR, 2017).

The RCC building process employs a relatively low-cement-content RCC for the inner portion of the dam, and the entire mass of RCC is normally encased in at least 0.30m of conventional concrete on both upstream and downstream faces, foundation, and upper portion of the dam with the current practice being the use of more RCC and less conventional concrete. The vertical joint spacings of monoliths are normally the same as those used in CMC dams (E.K. Schrader, as cited in E.G. Nawy, 2008).

Because of the beneficial characteristics such as lower costs, rapid construction, integral spillways and appurtenant structures, and lesser diversion and cofferdam, RCC construction methods have gained popularity and recently become an economically competitive alternative to CMC dams (USACE, EM 1110-2-2006, 2000).

(3) Grout-Enriched Roller-Compacted Concrete (GE-RCC) Facing system

For RCC gravity dams, there are several methods of facing systems (ACI 207.5R-11), but the two most popular systems are:

- (i) **Conventional method:** the conventional concrete facing is placed concurrently with each RCC lift;
- (ii) **GE-RCC method:** the RCC is placed against conventional formwork and then grout is applied to the uncompacted RCC mix adjacent to a formed face.

When utilized as a face for RCC dams, GE-RCC is a relatively new method that appears to hold a lot of potential. The construction process involves putting grout to uncompacted RCC adjacent to a formed face and vibrating the grout and RCC together by integrating the grout mix into the RCC mix with immersion vibrators to produce an in-situ conventional concrete face for an RCC dam at relatively cheaper cost than the conventional method of mixing and pouring each material separately and then vibrating them together.

2.3.4 Concrete Gravity Dams: Ethiopian Experience

According to the Ethiopian National Dams Inventory Report (2020) prepared by the Federal Democratic Republic of Ethiopia, Ministry of Water, Irrigation and Energy, dam construction in Ethiopia started in the late 1930's. The first dam built in the country is Aba Samuel dam, constructed on the Akaki River, tributary of the Awash River. At present, the country is estimated to have more than 300 operational and non-operational dams. Most of the dams are of embankment types – earthfill and rockfill dams. However, there has been no national or regional database that shows the exact number, operational and safety status of dams in the country.

Ethiopia is currently planning to build many more large, medium and small dams as part of the development and transformation plans as a step forward in the development of the large-scale hydropower potentials in the different regions of the country. Concrete gravity dams, especially RCC dams, are among the technologies that have been recently introduced in the construction of large dams for hydroelectric projects in Ethiopia. Among the recent developments include Gibe III dam, which is 243m high, located on the Omo river in the southern regional state, and currently regarded as the tallest RCC dam type in the world. The Grand Ethiopian Renaissance Dam (GERD), a 145m high RCC dam, located on the Abay river in the Benishangul Gumuz regional state in the north-western part of Ethiopia, is currently under construction, and will be the first largest dam in Africa when completed. Gibe IV (Koysha) dam, a 170m high RCC dam, is another milestone in the dam building process which is currently under construction and, when completed, will have a power generating capacity of 2160MW.

The table below shows summary of basic information (location, type, ownership, and size) about existing concrete gravity dams in Ethiopia.

Table 2-1: RCC and CVC dams in Ethiopia

S/ No.	Dam name	Type	Location	Year completed / Status	Height (m)	Reservoir storage ($\times 10^6 \text{ m}^3$)	Purpose	Owner	Installed capacity (MW)
1	GERD	RCC gravity	Benishangul-Gumuz Region	Under construction	175	74,000	Hydropower	EEP	6140MW
2	Gibe II (diversion)	CVC gravity	Oromia Region, Jimma zone	2010 /Operational	56	1.9	Hydropower	EEP	420MW
3	Gibe III	RCC gravity	Southern Region	2015 /Operational	243	15,245	Hydropower	EEP	1870MW

S/ No.	Dam name	Type	Location	Year completed / Status	Height (m)	Reservoir storage (x10 ⁶ m ³)	Purpose	Owner	Installed capacity (MW)
4	Koysha (Gibe IV)	RCC gravity	Southern Region	Under construction	170	6,540	Hydropower	EEP	2160MW
5	Tekeze	Concrete Arch	Tigray Region	2009 / Operational	188	9,310	Hydropower	EEP	300MW
6	Koka	Concrete gravity	Oromia Region, East Shoa zone	1960 /Operational	23.8	1,850	Hydropower	EEP	43.2MW
7	Aba Samuel	Concrete gravity	Oromia Region, East Shoa zone	1939/ Operational	22	65	Hydropower	EEP	3.0MW
8	Gefersa- I/II	Masonry and concrete gravity	Oromia Region, Finfine Zuria	1943 / Operational	18	8.49	Water supply	AAWSA	30,000m ³ / day
9	Gefersa- III	Masonry and concrete gravity	Oromia Region, Finfine Zuria	1966 / Operational	16	1.12	Water supply	AAWSA	30,000m ³ / day

[Source: National Dam Inventory Report; Ministry of Water, Irrigation & Energy, 2020]

Key: RCC: Roller-Compacted Concrete

CVC: Conventional Vibratory Mass Concrete

EEP: Ethiopian Electric Power

AAWSA: Addis Ababa Water and Sewerage Authority

Hence, the proper understanding of the basic design guidelines, design criteria, method of analysis and evaluations, initiations for the development of concrete gravity design and evaluation including seismic behaviours and responses is essential in the planning as well as implementation of similar projects in various regions of the country.

2.3.5 Historic Seismic Performance of Concrete Gravity Dams

According to USSD's report (2014), in the years before 2000, at least 100 concrete dams were shaken by earthquakes at a global level; however, only around 20 of these dams had measured or estimated peak ground accelerations (PGAs) of 0.20g or more. By the year 2013, about 20 dams have endured PGAs greater than 0.3g. Overall, concrete dam performance has been satisfactory, with minimal damage and only minor concrete spalling, cracking, or offsets between concrete blocks. No arch dam has ever sustained severe damage from an earthquake, however several of these structures have been subjected to considerable ground shaking.

The review of case studies of selected dams that have been shaken by more than 0.3g revealed that concrete gravity and buttress dams have been more seriously damaged by earthquakes than arch dams, and no arch dams have ever suffered significantly (USCOLD, 1992; USCOLD, 2000; USSD, 2014).

- (a) Blackbrook Dam, a 30m high concrete gravity dam, is the only dam in Great Britain documented to have suffered damage from an earthquake event that occurred in 1957 with a magnitude of 8 and at epicentral distance of 6.4 km from the location of the dam. The earthquake caused cracking of mortar joints of the downstream stone facing, lifting and toppling of the massive coping stones on both sides of the crest.
- (b) The Koyna Dam in India, a 103m high straight gravity dam, and the Hsinfengkiang Dam in China, a 105m high buttress dam, were both severely shaken by neighboring earthquakes of magnitudes 6.5 in 1967 and 6.1 in 1962 respectively. Both dams have suffered severe longitudinal cracking at the top. The Koyna Dam also had severe water leakage on the downstream face of the monoliths. The seismic design of the Koyna dam was made using the then-current standard procedures, which defined a horizontal seismic coefficient of 0.05 uniform along the entire height of the dam. The damage was directly linked to the underestimated design earthquake ground motions and inadequacy of the structure to withstand a higher level of ground shaking that would now be avoided in modern design and construction practices on similar structures. The two dams were renovated and are still operational. It is believed that lessons learnt from the seismic experience at Koyna Dam have contributed the most important information on the performance of gravity dams during earthquakes and thereby enhanced great motivation for study on this topic during the 1970s.
- (c) Sefid-Rud, Iran, a 106m high buttress dam, suffered severe cracking in the upper part of some buttresses and other forms of damage during the 1990 Manjil Earthquake (M 7.3).
- (d) Lower Crystal Springs Dam, in California, a 44 m high curved concrete gravity dam constructed of interlocking concrete blocks in California, was unharmed by the 1906 San Francisco earthquake of estimated magnitude M 8.3. An estimated 3m right-lateral slide was measured close to the major fault rupture, which was less than 183 meters from the dam. The same dam was once again shaken by considerable shaking as a result of the 1989 Loma Prieta earthquake, but remained unharmed.
- (e) Hoover Dam, located in Nevada, USA, is a 221m high curving gravity dam, is thought to be the origin of mild reservoir-triggered seismicity with an estimated magnitude of M 5.0 or less, but this did not impact the dam's integrity.
- (f) Several concrete gravity dams (Shih-Kang, Mingtan) were severely shaken during the 1999 Chi Chi Earthquake in Taiwan. These dams performed satisfactorily, with the exception of Shih-Kang Dam, which has undergone catastrophic earthquake-related failure: the Shih-Kang Dam, a 25m-high concrete gravity dam was destroyed by a fault rupture. The dam was 50 kilometers from the epicenter and had suffered from a PGA of 0.50g. Thrust movement of 8.8m vertically and 2.0m horizontally raised more than two-thirds of the dam. The foundation offset was responsible for

substantial damage in two spillway bays. Chi-Chi Dam, a concrete gravity dam, had experienced 0.50g PGA without thrust displacement and functioned without damage during the same occurrence.

- (g) Concrete gravity dams in Japan: Kasho dam (46.3m high), Sugawara dam (73.5m high), and Utsunomiya dam (14m high), all in Japan, experienced peak ground acceleration of approximately 0.5g, 0.16g and 1.0g, respectively during the 2000 Western-Tottori Prefecture Mw 6.7 Earthquake. However, they suffered no significant damage. The other concrete gravity dams - Takou dam, 77m high, and Miyatoko RCC dam, 48m high, have sustained the 2011 Tohoku Earthquake, Main shock M 9.0, having PGHA of 0.4g and 0.4g respectively.
- (h) Case histories of selected dam sites that have experienced significant earthquake shaking indicated that no significant damage has ever been suffered by an arch dam. However, concrete gravity and buttress dams have been, to date, affected more severely by earthquakes than have arch dams (USCOLD, 1992; USCOLD, 2000; USSD, 2014).

Typical case histories of selected concrete gravity dam sites that have been shaken by more than 0.3g are presented in Table 2-2 and Table 2-3 below (USSD, 2017).

Table 2-2 Summary of selected concrete dams shaken by PHGA > 0.3g and identified damages

Dam type	Number	Damage to dam	No damage to dam	Minor project damage
Gravity dams	9 CVC + 1 RCC = 10	1	5	4
Arch dams	5 CVC + 1 RCC = 6	1	2	3
Buttress dams	2	2	0	0
Spillways	1	1	0	0
Total	19	5	7	7

[Reproduced from USSD, 2017]

Table 2-3: Selected case histories of concrete gravity dams subjected to PHGA>0.3g

Dam name/year completed/country	Dam type / height	Earthquake	Damage rating / Observed physical damages	Remedies taken	Reference
Lower Crystal Springs Dam (1890), USA	concrete gravity dam, curved in plan (interlocking concrete blocks), 47m high	1906 San Francisco Earthquake, M 8.3; Peak acceleration: 0.52 - 0.68g	No slightest crack in the dam	None	USSD 2017, USCOLD 1992
Koyna dam (1963), India	Concrete gravity dam (straight), 103m high	1967 Koyna Earthquake, M 6.5 Peak accelerations @ lower gallery: 0.364-0.63g	Serious damage: horizontal cracks on either the upstream or the downstream face or on both faces of a number of non-overflow monoliths; cracks near change in downstream slope; significant leakage of water on downstream monoliths; relative movement between adjacent monoliths: overflow monoliths not damaged.	Dam rehabilitated by injecting epoxy resin, strengthened by increasing cross-sectional area and by prestressing systems. Damage was attributed to design or construction details that would be avoided in modern structures. The dam is still in service.	USSD 2017; USCOLD 1992
Bear Valley Dam (1912), (USA)	Concrete gravity dam (after converting multiple arch to concrete gravity dam type), 28m high	1992 Landers Earthquake, M 7.4 Peak Acceleration: 0.18g horiz, 0.08g vertical.	No damage to the dam after earthquakes; but only slight displacement of the girders on the highway bridge located at the crest of the dam.	Rehabilitated. Existing bridge on dam crest removed and replaced with new bridge being built on downstream end of dam crest.	USSD 2017 Annual Conference, USCOLD 2000
		1992 Bear Valley Earthquake, M6.6. Peak Acceleration: 0.57g horiz,, 0.21g vertical.			

Dam name/year completed/country	Dam type / height	Earthquake	Damage rating / Observed physical damages	Remedies taken	Reference
Gohonmatsu Dam (1900), Japan	Rubble concrete gravity dam, 33m high	1995 Kobe Earthquake, M 7.2, Peak acceleration: 0.83g	Structurally undamaged, but hairline cracks in the concrete capping of the parapet on the crest of the dam.	Repaired	USSD 2017; USSD 2014
Shih Kang dam (1977), Taiwan	Concrete gravity spillway dam (18 bays), 21m high	1999 Chi Chi Earthquake, M 7.6 Peak acceleration: 0.51g horiz., 0.53g vertical	Dam collapsed. Large displacements: left end of dam uplifted 11m; right side of dam uplifted 2.1m; diagonal offset of 7m. Damages: bridge girder collapsed; reservoir released discharge of 100-200m ³ /s; cracks along ogee crest, along lift lines at point of changes in geometry; dam separated from foundation	Repaired	USSD 2017; USSD 2014
Mingtian dam (1990), Taiwan	Concrete gravity Dam, 82m high	1999 Chi Chi Earthquake, M 7.6; distance 12kms; PGA 0.4-0.5g horizontal.	No structural damage to the dam body	None	USSD 2017 Annual Conference
Kasho dam (1989), Japan	Concrete gravity dam, 46.3m high	2000 Western Tottori Earthquake, M 7.3, Epic.distance: 4.8kms Peak acceleration @ lower gallery: horiz. 0.54g, vert.0.49g. At crest 2.09g (amplification x 3.87	No structural damage to the dam body; but minor damages to other structures. The main shock induced relative displacements: 2.8mm along dam axis; 0.7mm in the upstream direction; a resultant displacement 2.9mm. Walls cracked at base of control building at crest of dam.	Repaired	USSD 2017 Annual Conference, USSD 2014
Uh dam, Japan	Concrete gravity dam, 14m high	2000 Western Tottori Earthquake, M 7.3; epicentral distance 5.3kms; peak acceleration: horiz. 1.16g.	Although the dam was subjected to strong earthquake motion, the only damage to the dam was cracking with a width of 1 to 2 cm on the spillway chute near the base of the downstream slope.	Repaired	USSD 2017
Takou dam (2007), Japan	Concrete gravity dam, 77m high	2011 Tohoku Earthquake, Main shock M 9.0 PGA: 0.4g horizontal	Moderate damage: No damage to the main dam body; offset in parapet walls; cracked wall of the gate house; main shock Mw 9 caused power failure; Aftershock M 7.1 occurred and accelerograms recorded: PGAs:at foundations: upstream to downstream: 0.38g; cross-canyon 0.29g; vertical direction 0.27g;	Repaired	USSD 2017

Dam name/year completed/country	Dam type / height	Earthquake	Damage rating / Observed physical damages	Remedies taken	Reference
			Peak accelerations at the crest in the same directions were 1.79g, 2.04 and 1.33 g, respectively. Crest acceleration: upstream 1.79g, cross-canyon 2.04g, vertical 1.33g		
Miyatoko Dam, Japan	RCC gravity dam, 48m high	2011 Tohoku Earthquake, PGA 0.32g horiz.	No damage to dam reported.		USSD 2017
Sefid Rud Dam (Iran)	Concrete gravity buttress dam, 106m high	1990 Manjil Earthquake, M 7.7, PHGA 0.71 g	Moderate damage: horizontal cracks 18m below crest at change in geometry; minor upstream to downstream displacements between a few of the 23 blocks; leakage through some cracks; minor damage and displacement of all gates	Repairs included grouting of cracks and installing prestressed anchors.	USSD 2017

[Source: Reproduced from USSD, 2017; Khiavi, M.P., Gharabaghi, A.R.M. and K.Abedi, K., 2008)

USSD's (2014/2017) Conclusions and observations drawn from the selected case histories:

As seen in the above table, concrete dams have performed well against the effects of earthquakes. RCC dams performed no differently than dams built of conventionally. This may be due to the fact that concrete dams were likely built to greater design standards than some of the earlier embankment dams. Additionally, concrete dams are presumably less prone to aging, material deterioration, seepage, and inadequate maintenance. The real test for large concrete dams, though, will come when they are subjected to their design earthquake and have a full reservoir in a seismically active location. The collapse of Shih-Kang dam proved that concrete dams cannot be built to tolerate fault rupture.

The earthquake experience at Koyna Dam represents the most significant information on the performance of gravity dams during earthquakes, and provided much impetus for research on this problem during the 1970s. During earthquake, RCC dams performed no differently than dams built of conventionally placed concrete; horizontal cracks usually develop in the dam section at changes in stiffness. Amplification of accelerations was significantly increased from the base to the crest of the dams, for example in the cases of Kasho gravity dam (2.05g), Takou gravity dam (2.04g).

There could be a variety of explanations for why concrete dams have consistently behaved better than expected by design or study when subjected to earthquake ground shaking. The main reasons may be one or a combination of the following:

- The level of shaking has been too low to cause major damage (which means that most dams have not yet been tested to the anticipated hazard levels they have been designed for), and/or the duration of strong shaking may be too short to cause failure.
- Concrete dams are massive and redundant structures with considerable capacity to redistribute load effects when structural damage occurs in the structure.
- Usually, the duration of strong ground shaking is too short and it takes considerable time at high levels of shaking to cause structural failure.
- The additional safety margin due to the increase in dynamic tensile strength of concrete (usually taken as 1.5 times the static tensile strength) makes concrete dams stronger and more resilient during seismic ground shaking;.
- Damping behaviors is initiated in the body of the dam during the earthquake, increases as the concrete cracks and contraction joints open and close, and reduces the negative seismic impact on the dam.
- The contribution of the three-dimensional load distribution effects help in increasing the seismic stability of a dam and preventing failures, for example, as in the case of a gravity dam in a narrow valley.
- The entire potential seismic failure mode has not been fully achieved or experienced for concrete dams

2.3.6 RCC and CVC Dams: General Design Features, Considerations and Properties

2.3.6.1 Materials for Construction

The type of RCC materials are similar to those used in CVC, and include fine and coarse aggregates, water, cementitious ingredients (cement and pozzolan), and suitable admixtures (USBR, 2017).

RCC dams are regarded as the most recent and modern type of concrete dam construction that has largely superseded CVC as the conventional approach for gravity dam construction. While the situation is not the same for arch dams, it is clear that RCC gravity dams are now often thought to be the most competitive concrete dam type for many sites of dam construction.

2.3.6.2 Advantages/Disadvantages, Similarities and Differences

Both RCC and CVC concrete gravity dams necessitate the same level of foundation analysis and improvement. Furthermore, the fundamental method of structural analysis is the same. Their usual commonalities and differences are summarized below:

(a) Construction Methods:

- (i) CVC gravity dams are constructed in 1.5m to 2.3m thick blocks, whereas RCC is often installed in 0.3m thick compacted lifts spanning from one abutment to the other. Over each lift joint, a bedding mortar or bedding concrete is applied to ensure bonding and watertightness.

- (ii) Construction of an RCC dam is a relatively new and economical concept. When compared to CVC gravity dams, the utilization of high capacity placing and compaction equipment for RCC construction has resulted in the ability to place larger quantities of RCC faster and at a lower total cost (economic advantage). In addition, for dam rehabilitation projects, the cost of building spillways and embankment overtopping protection with layered, stepped RCC construction techniques may be less expensive than building conventional concrete overlays (USBR, 2017).

(b) Design Approach:

- (i) The design criteria for RCC dams are similar to those for CVC dams. RCC dams are also expected to behave similarly to CVC dams. RCC dams are currently appealing alternatives to CVC dams in most regions of the world due to such advantages as speed of construction, low unit prices of RCC, possible incorporation of spillway into dam body, and usage of typical dam shape. Despite these, there are some weaknesses or disadvantages with RCC dams such as watertightness under high hydraulic gradients, aging mechanisms, and unknown seismic performance, and also the design and construction techniques still being in the process of investigation and evolution (Wieland et al., 2004).
- (ii) The main design distinction between RCC dams and CVC dams lies in the assumptions and safety factors used to account for the uncertainty related to the bond on lifts and the possibility of reduced shear and tensile strength properties at the lift joint between successive layers of RCC (USBR, 2017). This is because RCC dams have significantly more construction joints than CVC dams as a result of the lift joints.
- (iii) Wieland, M., and Brenner, R. P. (2004) suggested that the dynamic behavior of CVC differs from RCC in that the material characteristics of CVC are isotropic within a concrete cantilever. In the case of RCC dams, the horizontal lift joints are spaced closely apart and span the whole dam length; the tensile strength of a lift joint may be considerably lower than that of isotropic mass concrete. This means that in the event of a large earthquake, any of these interfaces may experience the formation of horizontal cracks, which are the main sources of safety concerns. It is challenging to foresee when a monolithic, isotropic dam may begin to crack.

(c) Concrete Mixes and Properties:

- (i) Due to the broader range of aggregate properties utilized in RCC, the range of potential RCC properties may be wider than for CVC (USACE 2000, EM 1110-2-2006). Hardened RCC has properties that are similar to those of conventionally laid mass concrete. RCC dam building has utilized materials and gradations that would often be completely inappropriate for CVC and can even be advantageous. In terms of the ideal cementitious and pozzolan content, it is crucial to note that RCC does not adhere to the same guidelines and trends as conventional mass concrete (Nawy, E.G., 2008).

- (ii) Most of the gravity-type dams have relatively low stresses in most parts as a result of dead load and water load in the reservoir. The highly stressed areas are restricted to relatively thin strata at the dam's upstream and downstream sides, as well as at its heel and toe, where localized stress concentrations exist. As a result, a leaner RCC mix can be employed inside the internal body of the dams, where a relatively low strength is needed. High tensile stresses are often restricted to the up- and downstream surface layers, and can be brought on by earthquake action, especially in the top part of the dam. For these areas, a richer mix is typically applied to the outside surfaces for increased structural suitability, durability, and abrasion resistance (Wieland, M., and Brenner, R.P., 2004; USBR, 2017).
- (iii) The use of conventional cast-in-place concrete for both upstream and downstream facing elements in the case of RCC dams provides better appearance and aesthetic qualities (USBR, 2017). Many efforts have been made to consistently entrain air in RCC at facings by using Grout-Enriched RCC (GE-RCC). After the mix has been spread, a fluid grout is added to the RCC, with the intent being for the grout to mix uniformly throughout the RCC and turn it into a more fluid mix with higher cementitious content. The result is a zone of material (usually against a form at the face of the dam) that will have properties and appearance similar to conventional concrete. (Nawy, E.G., 2008).
- (iv) In the case of RCC dams, the use of conventional cast-in-place concrete for both upstream and downstream facing elements offers higher aesthetic attributes (USBR, 2017). By employing GE-RCC, a fluid grout is added to the RCC after the mixture has been spread, with the goal of causing the grout to mix evenly throughout the RCC and transform it into a more fluid mixture with a higher cementitious content. The end result is a zone of material (cast against a form at the dam's face) with characteristics and appearance similar to conventional mass concrete (Nawy, E.G., 2008).

(d) Permeability

- (i) Permeability testing of RCC has shown the RCC mass to be comparable to CVC of similar composition. The major concern for permeability of RCC structures has been seepage on horizontal lift lines and through vertical contraction joints or cracks (USBR, 2017). Several case studies of RCC dams have shown an overall good performance record for watertightness (USACE 2000, EM 1110-2-2006).
- (ii) The RCC mass has been demonstrated to be comparable to CVC of same composition by permeability testing. Seepage on horizontal lift lines and through vertical contraction joints or cracks have been the main causes of concern for permeability of RCC structure constructions (USBR, 2017). According to several case studies, RCC dams have performed well in terms of watertightness (USACE 2000; EM 1110-2-2006).

(e) Long term performance history

Worldwide, more than 161 RCC dams have been built by the end of 1994. Despite the widespread construction of RCC dams, more than 90% of them have been built since 1986. As a result, data on long-term performance have not yet been recorded. There are some performance lessons to be learned from the structures that are now in use, despite the fact that this is a relatively brief performance record. To date, the overall performance of RCC dams has been comparable to that of CVC dams in many respects (USACE 2000, EM 1110-2-2006).

(f) Thermal Properties:

- (i) In general, the thermal behaviors of CVC and RCC are similar. Both RCC dams and CVC dams have experienced cracking as a result of heat stresses. Due to the larger aggregate proportions in RCC, the coefficient of thermal expansion for RCC is typically slightly less than that of CVC. For both CVC and RCC, it ranges from 7 to $14 \times 10^{-6}/^{\circ}\text{C}$ (USACE 2000, EM 1110-2-2006).
- (ii) Due to the use of extremely wide spacing between transverse joints or even the absence of transverse joints, RCC dams have shown greater transverse cracking than CVC dams. According to USACE 2000 and EM 1110-2-2006, CVC dams have transverse construction joints that are tightly spaced and offer a decent amount of crack control.

(g) Bonding Between Lifts

- (i) **Shear Strength:** Both RCC and CVC dams must generally maintain proper bonding at lift lines to achieve needed factors of safety. This calls for both tensile strength and shear strength to withstand seismically generated vertical stresses as well as tensile strength to withstand hydrostatic water pressure. Resistance to horizontal forces across lift lines is provided by the cohesiveness of the bonded lift lines and the friction between the lower and higher surfaces. Most of the time, unbonded lift lines' friction resistance is insufficient to meet safety requirements, necessitating a real chemical bond (cohesion) between lifts (USBR, 2017).
- (ii) **RCC Lift lines:** Often, the design of RCC structures is dictated by the requirements for bonding lift joints for shear and tension rather than the design compressive strength requirement (USBR, 2017). However, the tensile (bond) strength at the lift joints for well proportioned, well-compacted RCC mix would be very close to the strength attained at the well-prepared lift joints of CVC (USACE 2000, EM 1110-2-2006).
- (iii) **CVC Lift lines:** The lift joint strength can be close to the strength of the parent concrete if proper construction techniques and quality control were used. According to laboratory studies of lift placement procedures, the strength of the lift joint is generally 72% of the parent concrete strength, if the top of the preceding lift is wet when the following lift is installed. The lift joint strength was 82% of the parent concrete strength if the preceding lift was placed first and given

time to dry (Tynes and McCleese, 1973, as cited in USBR, 2017). Field testing has revealed that the strength of the lift joints at Hoover Dam is roughly 92% that of the parent concrete. Low tensile strengths can exist where inadequate lift joint techniques exist. In the worst-case scenario, no connection is established between the lifts, and the tensile strength is zero.

2.3.6.3 Concrete Properties

(a) General

As stated in the USACE 2000, EM 1110-2-2006, in a hardened state, both RCC and CVC have properties that are similar to each other. The strength and elastic properties of RCC vary in much the same way that they do for CVC, depending on the mix components and proportions. Where variances do exist, they are usually attributed to RCC's lower water content, void content differences, or minor aggregate or other material differences. Due to the greater range of aggregate quality utilized in RCC, lower cementitious material contents, and the usage of significant volumes of mineral filler on some projects, the range of possible RCC properties may be wider than for CVC. RCC varies from CVC in that it has more horizontal planes of weakness (construction joints) during placement, each with lower tensile and shear strength than the parent concrete.

According to USACE (2007), Engineer Manual EM 1110-2-6053, the unit weight, compressive, tensile, and shear strengths, modulus of elasticity, and Poisson's ratio are the concrete properties and strengths that are crucial in the seismic design and evaluation of both conventional and roller-compacted concrete dams. Mass concrete has better properties under high rate loading than it does under static loading. As a result, the usage of concrete properties in the seismic analysis should take into account the dam's potential for high deformation rates and cyclic loading response. In general, the concrete's tensile strength and the propagation of tensile cracks determine how well a dam performs when subjected to seismic loads. However, it is important to consider the impact of lift joints when determining the actual tensile strength used to assess the dam's performance. The actual tensile strength across certain older dams' poorly constructed lift joints could be significantly lower than that of homogeneous concrete. As a result, it is critical that such weaknesses in mass concrete be accounted for in the seismic safety evaluation, and that material testing be conducted to determine the actual lower strength at lift joints. Concrete qualities for final design and evaluation should also be evaluated through testing to get material properties critical to structural analysis and thermal studies, as well as to validate in-place concrete strengths of both the parent concrete and lift joints.

(b) Strength Properties

(i) Compressive Strength

In both type of dams – CVC or RCC types, compressive strength is employed as a metric of the overall strength, as well as a good indicator of other properties including durability, mixture composition, and variability. In most cases, the tensile strength is the main design concern; and compressive strength requirement is rarely a design loading concern (USACE, 2000; USBR, 2017).

According to ACI 207.1R-05, mass concrete is rarely required to endure significant stress at an early age in mass concrete structures like CVC and RCC gravity dams. Therefore, the design strength is typically based on the strength at ages from 90 days to one year, and occasionally up to two years, to fully utilize the strength capabilities of the cementing ingredients and the significant growth in concrete strength with time. Although it may vary from one structure to another, this particular age is typically 365 days (USBR, 1976). The correlation factors at one year, where pozzolans are utilized to account for the ongoing strength development over 28 days, may vary from 1.15 to 3.0 times the strength of the wet-screened control specimens tested at 28 days. For mass concrete constructions, the required concrete strength is typically many times larger than the computed or predicted maximum design stresses.

The case studies presented in USACE 2000, EM 1110-2-2006, indicate that RCC mixtures may be designed for a minimum compressive strength of 13.8MPa for durability reasons alone. For seismically active regions, however, higher design compressive strength is often required in order to achieve the higher tensile and shear strength necessary. Typical compressive strengths of representative 14 RCC dams constructed between the years 1982-2006 in U.S. vary from as low as 8.6 MPa to as high as 48.1 MPa at an age of 1-year, with concrete mixes having nominal maximum size of aggregates in the order of 37.5, 50 and 75mm (ACI 207.1R-11), Typical compressive strengths of representative CMC dams constructed during the period 1942-1995 in U.S. vary from as low as 18.8 MPa to as high as 46.1MPa at an age of 1-year, with concrete mixes having maximum size of aggregates ranging from 100 to 225mm, predominantly of 150mm (ACI 207.1R-05).

(ii) Tensile Strength:

Numerous techniques, such as the direct tension method, the splitting tensile method, and the flexural test or modulus of rupture method, can be used to gauge the tensile strength of concrete. There are several connections between these tests and compressive strength. The splitting tensile test has been used more frequently to assess the parent tensile strength of concrete (USACE 2007, EM 1110-2-6053; ACI 207.1R-05).

Both RCC and CVC constructions are most vulnerable or weakest at lift joints. When RCC is suitably sized and compacted, its lift joints will have bond strengths that are similar to those at CVC lift joints.

Due to the decreased strength at construction joints and the impacts of strain rate, the ultimate tensile strength of the parent concrete (concrete without joints) determined by static load testing needs to be modified. It is unrealistic to anticipate that the bonding at construction joints will be the same as that of the parent concrete. According to Waterways Experiment Station, WES, 1973, as cited in USACE 2007, EM 1110-2-6053, the tensile strength of CVC lift joints cleaned by high pressure water jet is roughly 70% that of the parent concrete. When lift joints are adequately cleaned and filled with a mortar bedding, this relationship is also relevant to RCC joints (USACE 1995, EP 1110-2-12).

Tensile-to-compressive strength relationships:

- ACI 207.1R-05 provides an expression relating tensile strength, f_{dt}' , to compressive strength, f_c' , for mass concrete structures as follows:

$$f_{dt}' = 0.6f_c'^{1/2} \quad (2.1)$$

- According to the USBR's (2017) study based on collection of a wide range of data set of test results from 4 RCC dams and 18 CVC dams in the United States, the following tensile-compressive strength property correlations were determined for lift lines.

For RCC lift lines:

Splitting tensile Strength: $f_{st}' = 0.10*f_c'$ (2.2)

Direct tensile Strength: $f_{dt}' = 0.05*f_c'$ (2.3)

For CVC lift lines:

Splitting tensile Strength: $f_{st}' = 0.08*f_c'$ (2.4)

Direct tensile Strength: $f_{dt}' = 0.03*f_c'$ (2.5)

Eqs.(2.2) and (2.3) show higher estimates of splitting and direct tensile strengths for the data set of RCC lifelines than the corresponding values found for the CVC lift lines; however studies by Curtis et al. (2017) and USBR (2005) show the estimates are of similar magnitudes for both RCC and CVC lift lines

The USBR report (2017) suggests that the tensile-to-compressive strength empirical relationships, developed based on data set collected for a pilot study from a limited number of exiting RCC and CVC dams, showed were no clear correlations between the compressive/tensile strength parameters, and hence a number of trends could be noted. In the pilot study, the RCC dams have shown show higher ratios of splitting and direct tensile strengths to the uniaxial compressive strength and a higher ratio of splitting tensile to direct tensile strength than those values found in the CVC data set. Contrary to this trend, similar magnitudes for both RCC and CVC dams were estimated according to Curtis et al. (2017), as cited in USBR (2017) and USBR (2005).

- Canon (1995), as cited in USACE 2000, EM 1110-2-2006, suggested that the parent concrete tensile strength should be based on direct tensile test strengths or, at most, 75% of splitting tensile strengths. Based on historical data, he further developed the empirical tensile-compressive

strength correlations depicted in Table 2-4 and Table 2-5 to estimate the tensile strength of the lift joints for RCC and CVC structures based on splitting tensile tests performed on parent RCC with workable consistency and the usage of bedding mortars between the lifts. The estimated lift joint strength values span a wide range, with lower values used for uncrushed, low strength aggregates and unbedded lift joints and higher values for crushed, high strength aggregates and bedded lifts.

Table 2-4 Empirical relationship for estimating RCC lift joint direct tensile strength, workable consistency (Canon, 1995)

Characteristic compressive strength, f_c' (365 days)	Nominal maximum size of aggregates (NMSA)	Split tensile strength of parent concrete (cylinders), f_{st}'		Direct tensile strength of lift joints, f_t'	
		Min.	Max.	Min.	Max.
(MPa)	(mm)	(MPa)	(MPa)	(MPa)	(MPa)
≤ 24.1	$\leq 75\text{mm}$	$0.08f_c'$	$0.17f_c'$	$0.03f_c'$	$0.06f_c'$
	$> 75\text{mm}$	$0.08f_c'$	$0.17f_c'$	$0.025f_c'$	$0.055f_c'$

[Source: Reproduced from USACE 2000, EM 1110-2-2006]

Table 2-5 Empirical relationship for estimating CVC lift joint direct tensile strength (Canon, 1995)

Characteristic compressive strength, f_c' (365 days)	Nominal maximum size of aggregates (NMSA)	Split tensile strength of parent concrete, f_{st}'		Direct tensile strength of lift joints, f_t' (MPa)	
		Min.	Max.	Min.	Max.
(MPa)	(mm)	(MPa)	(MPa)	(MPa)	(MPa)
≤ 20.7	$\leq 75\text{mm}$	$0.10f_c'$	$0.15f_c'$	$0.055f_c'$	$0.0085f_c'$
	$> 75\text{mm}$	$0.10f_c'$	$0.15f_c'$	$0.05f_c'$	$0.075f_c'$

[Source: Reproduced from USACE 2000, EM 1110-2-2006]

- According to Raphael's study (1984), the ratio of the apparent dynamic tensile strength to the static tensile strength, together with other characteristics, will be utilized to evaluate the performance of existing concrete gravity dams for the linear-elastic time-history analysis. Because the concrete's apparent dynamic tensile strength is twice as great as its static tensile strength, it can withstand stresses up to double the static amount (Figure 2-2). When evaluating the outcomes of a linear dynamic analysis, the stress demand associated with a demand-to-capacity ratio of 2 is equal to the so-called "apparent" dynamic tensile strength of the concrete, i.e. $f_t' = 0.64 * f_c'^{2/3}$.

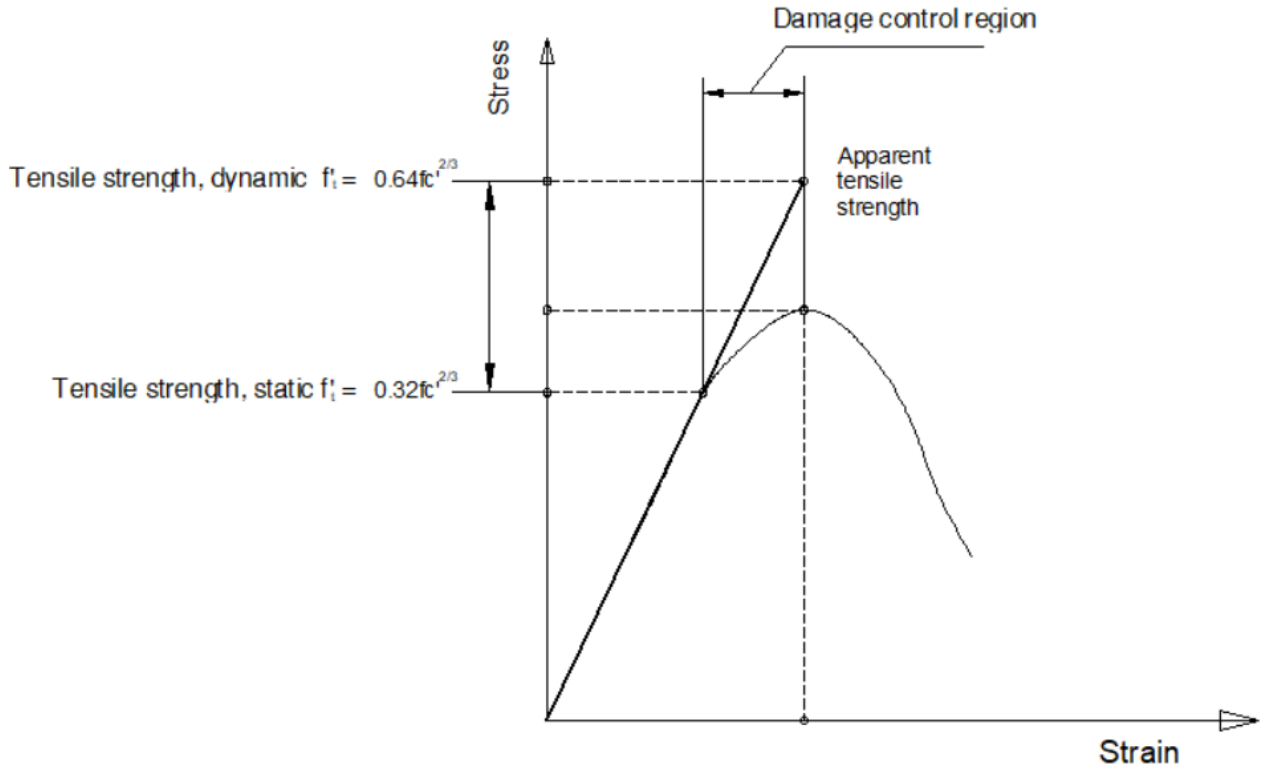


Figure 2-2 Apparent tensile strength of concrete for seismic response evaluation (USACE, 2003)

(iii) Shear Strength:

Shear strength of mass concrete structures is another most crucial concrete design parameter used in structural analysis and evaluations. Shear strength is a function of internal friction that varies with normal compressive stress and cohesive strength that should be assessed using shear strength tests carried out at various normal stress levels that cover the range of normal stresses to be anticipated in the dam. The shear strength vs normal stress curve should be created using these parameters (USBR, 1976).

The Mohr envelope relationship, which combines frictional resistance and cohesion (bond), may typically be used to represent shear strength. According to ACI 207.1R-05 (2012), the construction joints between lifts of RCC and CVC experience the least amount of shear within the structure.

$$\tau = C + \sigma_n \tan \phi' \quad (2.6)$$

where: τ = shear strength (MPa))

C = cohesion (MPa)

σ_n = normal or confining stress (MPa)

ϕ' = internal friction angle (deg.)

USACE (2000) Guidance, EM 1110-2-2006:

For the purpose of preliminary design, shear strength of parent concrete is usually derived from historical data or tests conducted on several dams of similar types, and then modified to simulate the lift joint shear strength. However, final design shear strength values for moderate to large dams in a seismically active regions, are required to be developed through laboratory testing of test section cores.

– Parent concrete shear strength (based on study of test results obtained from cores):

Based on study of test results of shear strength from cores of several concrete dams, the shear strength properties for RCC are found to be similar to those for CVC. Cohesion values for the parent RCC have ranged from as little as 0.5 MPa to 4.1 MPa (McLean and Pierce 1988, as cited in USACE 2000). On the other hand, cohesion values for a workable parent RCC have ranged up to 20 percent of the compressive strength, and their friction angles from 40 to 60 degrees.

– Lift joint shear strength (based on study of test results obtained from cores):

In general, the shear strength at the lift joints is the key parameter for design. According to the study and evaluation of shear strength from cores of many concrete dams, RCC shear strength for lift joints can be lower than CVC and more variable on some projects. The values of C have ranged from $0.03 \cdot f_c'$ to $0.06 \cdot f_c'$ for lift joints without beddings. C has ranged from $0.09 \cdot f_c'$ to $0.15 \cdot f_c'$ for bedded lift joints. The friction angle for both bedded and unbedded lift joints has remained virtually constant (McLean and Pierce (1988), as cited in USACE 2000). For preliminary design purpose, $C = 0.05 \cdot f_c'$ may be recommended for lift joint surfaces with mortar-beddings; and a value of 0 should be assumed if no bedding is provided. Similarly, for preliminary design or minor projects, a value of $\phi' = 45$ degrees can be considered for both parent and lift joint shear strength. Assumed values must be validated for final design by testing on lab-prepared samples and cores retrieved from test fills.

USBR (2000) Guidance:

USBR (2006) recommends using shear strength properties established for the structure under consideration for the numerical analysis of critical structures due to the inherent variation of properties in different structures, and properties may vary due to the manufacture of the construction materials, time effects, spatial differences in the placement of the material, or a variety of other reasons. Typical shear strength parameters that can be used in early (preliminary) assessments are:

Intact (parent) concrete:	Cohesion, $C = 0.10 \cdot f_c'$; friction angle, $\phi' = 45$ degree
Bonded lift lines:	Cohesion, $C = 0.085 \cdot f_c'$; friction angle, $\phi' = 45$ degrees
Unbonded lift lines:	Cohesion, $C = 0.34 \text{ MPa}$; friction angle, $\phi' = 40$ degrees

Concrete-Rock Interface Shear Strength (USBR, 2006):

Shear strength is a key parameter in the analysis of dam's sliding stability. Bonded surfaces have high cohesion and friction. Bonded and unbonded strengths on a particular surface may not be mobilized simultaneously. USBR (2006) assumes a slide plane with 0.34MPa apparent cohesion and 40° friction angle for preliminary stability analyses in the absence of shear strength test results.

(c) Elastic Properties

(i) Modulus of Elasticity (E_c)

The modulus of elasticity (E_c) is defined as the ratio of normal stress to corresponding strain below the proportional limit of the elastic range. Since concrete lacks true elastic properties, the stress-strain relationship for a constantly increasing load typically takes the shape of a curved line. For practical reasons, however, the modulus of elasticity is taken into account to be a constant for the majority of stresses to which mass concrete is typically exposed (USACE 1995, EM 1110-2-2200).

According to USACE 2000, EM 1110-2-2006, properly proportioned and consolidated RCC should have an elastic modulus equal to or greater than that of CVC produced with materials that have a comparable compressive strength. Modulus of elasticity is highly influenced by the modulus of elasticity of the aggregate (ACI 207.1R-05, 2012). The tensile modulus of elasticity is generally assumed to be identical to the compressive modulus of elasticity. The values of E_c for RCC tend to have a wider range than E_c for CVC. The same modulus-strength relationships used for CVC may be used for RCC (USACE 2000, EM 1110-2-2006).

(ii) Poisson's Ratio (ν)

Poisson's ratio is the measure of ratio of the lateral to longitudinal strain resulting from an evenly distributed axial stress applied on the structural component. Poisson's ratio values for RCC are often comparable to those for CVC mixes. For static analysis, values for the majority of RCC dams have ranged from 0.17 to 0.22, with 0.20 being recommended in the absence of testing. When assessing stresses caused by seismic loads, because Poisson's ratio is strain-rate sensitive, the static value may be lowered to 70% of the static value (ACI 207.1R-05; USACE, 2000). Standard tests like the ASTM C 469, "Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression," are used to determine the modulus of elasticity and Poisson's ratio.

(iii) Creep

Creep is characterized as time-dependent strain (deformation) of a structural element by a constant load. Creep is a function of the material properties and proportions, is strongly related to the concrete compressive strength and elasticity modulus. Based on several tests, Creep rate values for RCC have

ranged from 1.5 to 29 millionths per MPa (USACE, EM 1110-2-2006, 2000). When design requires more exact values, creep should be based on the standard test for creep of concrete in compression (such as ASTM C 512 or other accepted standards).

(d) Dynamic Properties:

(i) Dynamic tensile and compressive strengths:

Raphael (1984) discusses how dynamic loading affects concrete's tensile strength. Concrete's tensile strength, like its compressive strength, is strain-rate sensitive, and he recommended that the direct tensile strength be increased by a factor of 1.50 to be the equivalent of dynamic tensile strength under high strain-rate loading. This correction factor is applicable to both the parent material and the lift joint tensile strength (Cannon, 1995; Raphael, 1984, as cited in USACE 2000, EM 1110-2-2006).

According to Harris et al., 2000, as cited in ACI 207.1 R-05, the ultimate compressive and tensile strength as well as the elastic modulus of CVC all tend to rise under conditions of rapid dynamic loading, in accordance. Results of testing on dynamic material properties are not yet available for publication. Recent tests on CVC cores have shown that the dynamic compressive strength, dynamic modulus of elasticity in compression, and dynamic splitting tensile strength all vary between 75 and 145% of the static strength.

(ii) Dynamic Poisson's ratio

Poisson's ratio is also strain-rate sensitive and the static value may be reduced by up to 30 percent when evaluating stresses due to seismic loads (Bruhwiler 1990, as cited in USACE, 2000). Some testing has suggested that Poisson's ratio is not significantly sensitive to the strain rates normally considered for mass concrete dams (Hess 1992, as cited in USACE, 2000).

(iii) USACE (2007) Recommended Static / Dynamic Relationships

As per the guidelines of USACE 2007, EM 1110-2-6053, both strength and elastic properties are strain rate sensitive, and in the absence of test data, the following relationships between static and dynamic properties may be assumed.

Dynamic modulus	= 1.15 x Static Modulus
Dynamic Poisson's Ratio	= 0.70 x Static Poisson's ratio
Dynamic compressive strength	= 1.15 x static compressive strength.
Dynamic tensile strength	= 1.50 x static tensile strength

(iv) USBR (2006) Study

Based on a number of static and dynamic tests conducted at in-situ moisture contents of various dams, USBR (2006) suggests that the ratio of dynamic-to-static modulus can range from 0.70 to 1.20 with the overall average heading toward 1.0. It is further recommended that if the concrete dynamic modulus is to be determined from laboratory tests, i.e., if the dynamic modulus is not known (i.e. no tests available), the static laboratory modulus obtained from laboratory test is used for both the static and the dynamic modulus.

(e) Raphael's recommendation (1984)

Raphael (1984) discusses the effects of dynamic loading on the tensile strength of concrete. Like compressive strength, tensile strength of concrete is strain-rate sensitive. Dynamic tensile strength, during high strain-rate loading, is considered equivalent to the direct tensile strength multiplied by a factor of 1.50. This adjustment factor applies to both the tensile strength of the parent material and to the tensile strength at the lift joints (Cannon, 1995; Raphael, 1984, as cited in USACE 2000, EM 1110-2-2006).

When the tensile stress in existing dams exceeds 150 percent of the modulus of rupture, it is recommended that nonlinear analyses be carried out to evaluate the extent of cracking (USACE 1995, EM 1110-2-2200).

(f) Thermal Properties

In most RCC dams and CVC dams, it is generally necessary to investigate thermal properties of the mix to assess the effects of stresses induced by temperature changes in the concrete and to determine the temperature controls necessary to avoid undesirable cracking (USBR, 2017).

Thermal properties, including specific heat, conductivity, diffusivity, coefficient of thermal expansion, and adiabatic temperature rise, are of primary concern for both conventional and roller compacted mass concrete, and are governed by the thermal properties of the mixture constituents (ACI 207.1R-05, 2012).

The coefficient of thermal expansion is usually slightly smaller for RCC (because of higher aggregate content) than for CVC. The coefficient of thermal expansion for CVC and RCC varies between 7 and 14 millionths per oC. A value of 9 millionths per oC can be used for preliminary RCC design studies (USACE 2000, EM 1110-2-2006).

(g) Permeability:

RCC permeability is mostly determined by voids in the compacted mass as well as the porosity of the mortar matrix, and is thus virtually entirely determined by mixture proportioning, placement method, and degree of compaction. In general, an unjointed mass of RCC proportioned with enough paste has permeability values comparable to CVC. For well-compacted, usable RCC mixtures, test values typically range from 0.15 to 15 10^{-9} cm/s (ACI 207.1R-05, 2012).

2.3.6.4 Foundation Rockmass Properties

The modulus of deformation, which describes the relationship between applied load and the resulting elastic plus inelastic deformation, characterizes the flexibility of the jointed rock foundation. The effective modulus of rock mass deformation takes into account all joints, shears, and faults as well as the rock matrix. In order to account for natural rock discontinuities, the foundation deformation modulus is best approximated or evaluated in situ. It can also be estimated using an appropriate reduction factor from the elastic modulus of the rock.

In situations where the foundation is composed of different rock strata at different levels, different modulus values may need to be utilized and the foundation may need to be analyzed as a composite of two or more layers. The Young's modulus of elasticity (E_f) and the modulus of deformation (E_d) are equivalent terms in a linear-elastic analysis (USACE 1995).

Although Poisson's ratio for intact rock specimens can range from less than 0.1 to more than 0.35, it is typically estimated to be in the range of 0.2 to 0.3 for most foundation rock masses. The value chosen for the Poisson's ratio within this range often has little impact on the analysis findings (USBR, 2006).

2.3.6.5 Properties of Incompressible Fluid (Water) Elements

According to the USBR (2006), one method of including reservoir loads is to model the added mass of water with finite elements and use material properties which represent the behavior of the water. The following are typical recommendations for properties of a linearly elastic incompressible reservoir (water):

Modulus of elasticity of water	: $E_w = 1.31 \text{ MPa}$
Density	: $\rho_w = 1000 \text{ Kg/m}^3$
Poisson's ratio	: $\nu = 0.4999$
Bulk modulus	: $K = 21,800 \text{ MPa}$

2.4 Seismic Design and Evaluation Criteria

2.4.1 General Requirements

According to the USACE (2016) guidance, ER 1110-2-1806, seismic studies must include the seismotectonic, geologic, site, geotechnical, and structural investigations necessary to properly select the design ground motions and assess how the foundation and structures will respond to any earthquake events that may occur at the project site.

The design of new dam is different from the seismic evaluation or reevaluation of existing dam features. The evaluation process starts with an examination of the project's foundation conditions, construction materials, and project features. Additionally, knowledge of the construction techniques employed at the time the project was constructed is required. It is necessary to screen existing project features for sufficient seismic capacity using the available data, including geological maps, boring logs, acceleration contour maps, standard response spectra, structural and geotechnical analysis and studies, construction photographs, and as-built project records.

For projects in high or seismic hazardous areas where seismic loading influences the design, detailed site explorations, site specific ground motion studies, and structural evaluations should be conducted. Governing loading conditions must be determined by evaluating applied loads and projected performance (USACE 2016, ER 1110-2-1806).

2.4.2 Approaches to Developing Resposnse Spectra

Earthquake ground motions are commonly described in terms of response spectra and acceleration-time histories for the design of new dams as well as for evaluation of existing ones. The response spectra and acceleration-time histories are created using two methods: standard seismic studies and site-specific seismic studies.

2.4.2.1 Standard Response Spectra

This approach involves studies based on existing general seismological studies, site data and information, and simplified evaluation methodologies (USACE 2016, ER 1110-2-1806). Standard response spectra are derived from the national seismic hazard maps of a country or areas and are based on spectral shapes and spectral acceleration ordinates produced for recorded ground motions with similar subsurface properties. The effective peak ground accelerations or the spectral accelerations acquired from the national seismic hazard maps are used to define the standard spectral forms. The standard response spectra can be used as a starting point for conceptual design development and evaluations, determining whether the earthquake loading controls the design, determining the need for more in-depth analysis, and determining the potential impact of the earthquake loading on construction costs (USACE 2007, EM1110-2-6053).

In general, preliminary values of ground motions collected from published and online national regional spectral acceleration maps are used in developing standard response spectra. The approaches are suitable for preliminary and screening studies in all seismicity zones, and may be adequate for final design or evaluation in low to moderate seismic hazard zones (USACE 2016, ER 1110-2-1806).

2.4.2.2 Site-Specific Response Spectra

This method is applied to assess the risks associated with the project and how the project features respond to seismic loads based on the actual site and structural conditions, source characteristics, source-to-site distance, depths, transmission path properties. Thus, it is essential to carefully organize and carry out comprehensive field exploration and testing operations comprising defining the project geological, seismological and geotechnical settings; identifying seismic sources that might have an impact on the project site; and figuring out how the foundations and structures will respond. Any design or evaluation decisions as well as the design of any corrective actions must be based on the final findings of site-specific investigations. For all projects with high or moderate seismicity, for which earthquake loading governs the design, site-specific assessments must be carried out. As a result, it is commonly understood that site-specific research will yield more reliable acceleration spectra than using codified standard acceleration spectra (USACE 2007, EM 1110-2-6053).

The two main methods for conducting site-specific seismic hazard studies are:

- Deterministic seismic hazard analysis (DSHA)
- Probabilistic seismic hazard analysis (PSHA)

(a) Deterministic Seismic Hazard Assessment (DSHA)

The DSHA approach estimates discrete, single-valued events or models of ground motion at the site using historical seismic and geological data as well as the known seismic sources that potentially affect the location. Usually, the magnitude and location of one or more earthquakes are reported in relation to the site. Typically, it is assumed that earthquakes originate at the source nearest to the site. The magnitude, source-to-site distance, source mechanisms, and site conditions are used to deterministically estimate the site ground motions. The Maximum Credible Earthquake (MCE) ground motion parameters are estimated using the standard procedures of DSHA, and the MCE does not have a return period (USACE 2016, ER 1110-2-1806).

(b) Probabilistic Seismic Hazard Analysis (PSHA)

The PSHA technique uses probabilistic theories to forecast the likelihood that a specific level of ground shaking will occur at a site over a given time span by taking into account the uncertainties associated with earthquake size, location, frequency, and predictive relationships (Datta, T. K., 2010). Inherent uncertainties and the probability (frequency of occurrence) of various magnitude earthquakes on each

significant seismic source are investigated under this approach, according to which the acceptable probability of exceedance during the course of the structure's service life for a specific return time would be utilized the findings of a PSHA are utilized to determine the site ground motions (USACE 2016, ER 1110-2-1806).

The PSHA technique uses probabilistic theories to forecast the likelihood that a specific level of ground shaking will occur at a site over a given time frame by taking into account the uncertainties associated with earthquake size, location, frequency, and predictive relationship (Datta, T. K., 2010). Inherent uncertainties and the probability of various magnitude earthquakes on each significant seismic source are immediately taken into account in the study. According to the acceptable probability of exceedance during the course of the structure's service life for a specific return period, the findings of a PSHA are utilized to determine the site-specific ground motions (USACE 2016, ER 1110-2-1806). The primary distinction between DSHA and PSHA is that, whereas the deterministic method gives a single estimate of the peak ground acceleration and response spectral amplitudes, the probabilistic method gives these estimates as a function of the probability of exceedance or return period.

The details of the above two approaches – DSHA and PSHA - are beyond the scope of this study and hence not well covered in this study.

2.4.3 Design /Evaluation Earthquake Ground Motions

2.4.3.1 Design / Evaluation Earthquakes: ICOLD (2010) Approach

It is necessary to define "design earthquakes" that represent the suitable seismic parameters (e.g., magnitude, acceleration, spectral ordinates, duration, etc.) to be employed in the design of new dams or seismic performance assessment of existing dams. The main body of concrete gravity dams and its structural components are evaluated for their seismic performance under the following design earthquakes (ICOLD 2010, Wieland, 2012):

- (a) Maximum Credible Earthquake (MCE):** The MCE is the event that, given the region's seismic history and seismotectonic configuration, would yield the maximum predicted ground motion at the dam site. It is estimated using the deterministic earthquake scenarios in which no return period is defined and the 84 percentiles (mean plus one standard deviation) should be used as the ground motion parameters for the MCE.
- (b) Maximum Design Earthquake (MDE).** The MDE ground motion parameters are estimated using the PSHA approach for a return period of 10,000 years for big dams. Shorter return periods can be chosen for dams with minimal or limited damage potential. The mean values of the MDE ground motion parameters must be taken. In situations where a single seismic source (fault) makes up the majority of the seismic hazard, uniform hazard spectra can be employed for the seismic design or

evaluation. Otherwise, different scenarios for earthquakes may be established based on the deaggregation of the seismic hazard (magnitude versus focal distance).

- (c) **Safety Evaluation Earthquake (SEE).** The SEE is the earthquake ground motion that a dam must be able to withstand in order to prevent an uncontrolled reservoir release. For big dams, most unfavorable ground motions resulting from either MCE or MDE can be considered as SEE. The SEE must be at least equal to the MDE if it is impossible to realistically assess the MCE. In order for the dam and other safety-relevant components to function after the SEE, the SEE serves as the governing earthquake ground motion for safety assessments and seismic design.
- (d) **Operating Basis Earthquake (OBE).** The OBE is the earthquake ground motion that may be anticipated to happen throughout the dam's existence during which there must be no damage or interruption of the normal services of the dam. With a 100-year service life, there is a 50% chance that it will happen. The OBE ground motion parameters are estimated based on PSHA for an assumed return period of 145 years, and the mean values of the ground motion parameters can be taken.
- (e) **Reservoir-triggered seismicity (RTS).** RTS is a seismic event that may be caused by the incremental loading effects of reservoir and the buildup of pore-water pressure. Past experience revealed that RTS has primarily happened in dam constructions in dam sites where the maximum reservoir depth was greater than 100 meters or the reservoir volume was greater than 500×10^6 m³. Since RTS typically occurs within a few years of the reservoir being impounded, it affects the seismic hazard and design criteria for ancillary structures, buildings, and infrastructure, but not the ground motion of the SEE, which is used to design and test the seismic safety of dams. Today, it is widely acknowledged that substantial RTS can only happen in areas with strong tectonic stresses in the earth's crust, where increased gravity stresses and the propagation of pore pressure from reservoir impounding can cause release of seismic energy (Wieland, M. 2017; Aman, A., 2020).

2.4.3.2 Characterization of Design /Evaluation Earthquakes

The different design earthquakes are characterized by the following seismic parameters (Wieland, M., 2012).

- (i) Peak ground accelerations (PGAs): horizontal and vertical earthquake components.
- (ii) Typical damping ratio of 5% is mostly assumed.
- (iii) Acceleration response spectra: horizontal and vertical earthquake components
 - For MCE: the 84 percentile values of acceleration spectra obtained from the DSHA method using different attenuation models.
 - For MDE and OBE: uniform hazard spectra obtained from the PSHA method (mean values); and

(iv) Spectra-compatible acceleration time histories: horizontal and vertical components

- For MCE: ground motions that are determined either from a random process or
- For SEE, MDE and OBE: by scaling or spectral matching of recorded (real) earthquake ground motions.
- Artificially generated acceleration-time histories of the horizontal and vertical earthquake components

2.4.3.3 Developing Design (Target) Response Spectra

(a) Definitions

Peak ground acceleration (PGA). PGA is the maximum amplitude of the ground acceleration time history during earthquake ground motions. This corresponds, in terms of structural response, to the absolute value of the absolute acceleration of a single degree of freedom (SDOF) system with infinite stiffness (i.e. with a natural period of vibration equal to zero).

Response Spectrum. A response spectrum is a graphic presentation of plots showing the peak values of the structural responses (displacement, velocity, or acceleration) to a given seismic input for a variety of single degree of freedom (SDOF) systems with various natural vibration periods. As a result, the peak accelerations that a group of SDOF systems with various natural periods may display for a certain component of ground motion are represented by an acceleration response spectrum.

(b) Developing Design Response Spectra

As discussed in section 2.4.2 there are two basic approaches for the development of design response spectra: site-specific procedure or standard procedure. The development of design response spectra, according to the USACE (2007) guidance and the Ethiopian code provisions (ES EN 1998-1:2015), are discussed below.

(b1) Standard Response Spectra: USACE (2007) Guidance, EM 1110-2-6053

The following are some parameters to take into account and steps to implement while developing a standard response spectrum:

❑ Seismic hazard data

The PGA calculated using the PSHA technique and the spectral ordinates at periods of 0.2 seconds (denoted by S_s) and 1.0 seconds (denoted by S_1) should be computed for the predicted probabilities of exceedance for a given site location. The shear-wave velocity in the top 30 meters of hard rock sites, which are used to determine ground motion estimates, is approximately 760 m/sec.

❑ Site Conditions

The response of the structure may be significantly impacted by the influences of the soil and foundation conditions. To reflect site characteristics, the shape of the standard response spectra can be modified to account for these site impacts by scaling the spectral ordinates to the appropriate values for other local variables using site coefficients. The values of the two coefficients, S_s and S_1 , which were determined for firm rock conditions (defined by a shear-wave velocity of approximately 760 m/s in the top 30 m) should be scaled by the site coefficients, F_a and F_v , as calculated by.

$$\begin{aligned} S_s &= F_a \cdot S_a \\ S_1 &= F_v \cdot S_1 \end{aligned} \quad (2.7)$$

Table 2-6 Site classification

Site class	Description	Shear wave velocity, V_s (m/s)	Reference
A	Hard rock	1500	USACE (2007), EM 1110-6053
B	Rock	750 - 1500	

Table 2-7 Site correction coefficient, F_a

Site class	Coefficient, F_a					Reference
	$S_s \leq 0.25$	$S_s=0.5$	$S_s=0.75$	$S_s=1.00$	$S_s \geq 1.25$	USACE (2007), EM 1110-6053
A	0.8	0.8	0.8	0.8	0.8	
B	1.0	1.0	1.0	1.0	1.0	

Table 2-8 Site correction coefficient, F_v

Site class	Coefficient, F_v					Reference
	$S_s \leq 0.10$	$S_s=0.2$	$S_s=0.30$	$S_s=0.4$	$S_s \geq 0.50$	USACE (2007), EM 1110-6053
A	0.8	0.8	0.8	0.8	0.8	
B	1.0	1.0	1.0	1.0	1.0	

❑ Correction for damping ratio

A 5% inherent structural dampening is used as the basis for the seismic risk information. If spectral values for alternative damping ratios are required, the standard spectral accelerations must be modified using the correction coefficients, D_s and D_1 , as given in in Table 2-9 below.

Table 2-9 Damping correction coefficients, D_s and D_1

Damping (%)	D_s	D_1	Refererence
5	1.0	1.0	USACE (2007), EM 1110-6053
6	1.06	1.04	
7	1.12	1.08	

❑ Constructing the standard horizontal response spectrum (Figure 2-3 (a))

The following equation can be used to estimate the values of the periods defining the interval of maximal spectral amplification:

$$\begin{aligned} T_s &= \frac{D_s S_1}{D_1 S_s} \\ T_0 &= \frac{1}{5} T_s \end{aligned} \quad (2.8)$$

where: T_0 = the start of maximum amplification plateau,

T_s = the maximum amplification plateau

The standard horizontal spectrum is then defined as follows:

$$S_a(T) = \begin{cases} S_s \left(\left(\frac{5}{D_s} - 2 \right) \frac{T}{T_s} + 0.4 \right) & \text{for } 0 \leq T < T_0 \\ \frac{S_s}{D_s} & \text{for } T_0 \leq T < T_s \\ \frac{S_1}{D_1 T} & \text{for } T \geq T_s \end{cases} \quad (2.9)$$

❑ Constructing the standard vertical response spectrum (Figure 2-3 (b))

▪ Vertical-to-horizontal response spectral ratio

According to USACE (2007), EM 1110-6053, the commonly used vertical to horizontal response spectral ratio of 2/3 may be significantly exceeded at short periods for near-source distance conditions, and thus adjustment should be made taking the source-to-site distance (R) into account by applying the factors listed in Table 2-10. In most preliminary design scenarios, the source-to-site distance will be unknown, and $R = 25\text{km}$ may be assumed. This approach is applicable to both standard and site-specific spectra.

Table 2-10 Adjustment factor, F_v , for vertical response spectrum

Source-to-site distance (km)	Conversion Factor, F_v	Refererence
≤ 10	1.0	USACE (2007), EM 1110-6053
25	0.84	
> 40	0.67	

▪ The spectral period defining the upper limit of the maximum vertical amplification plateau, T_{sv} .

$$T_{sv} = 0.67 \frac{T_s}{F_v} \quad (2.10)$$

The standard horizontal and vertical response spectra (USACE, 2007) are presnted schematically in **Figure 2-3 (a) and (b)** below:

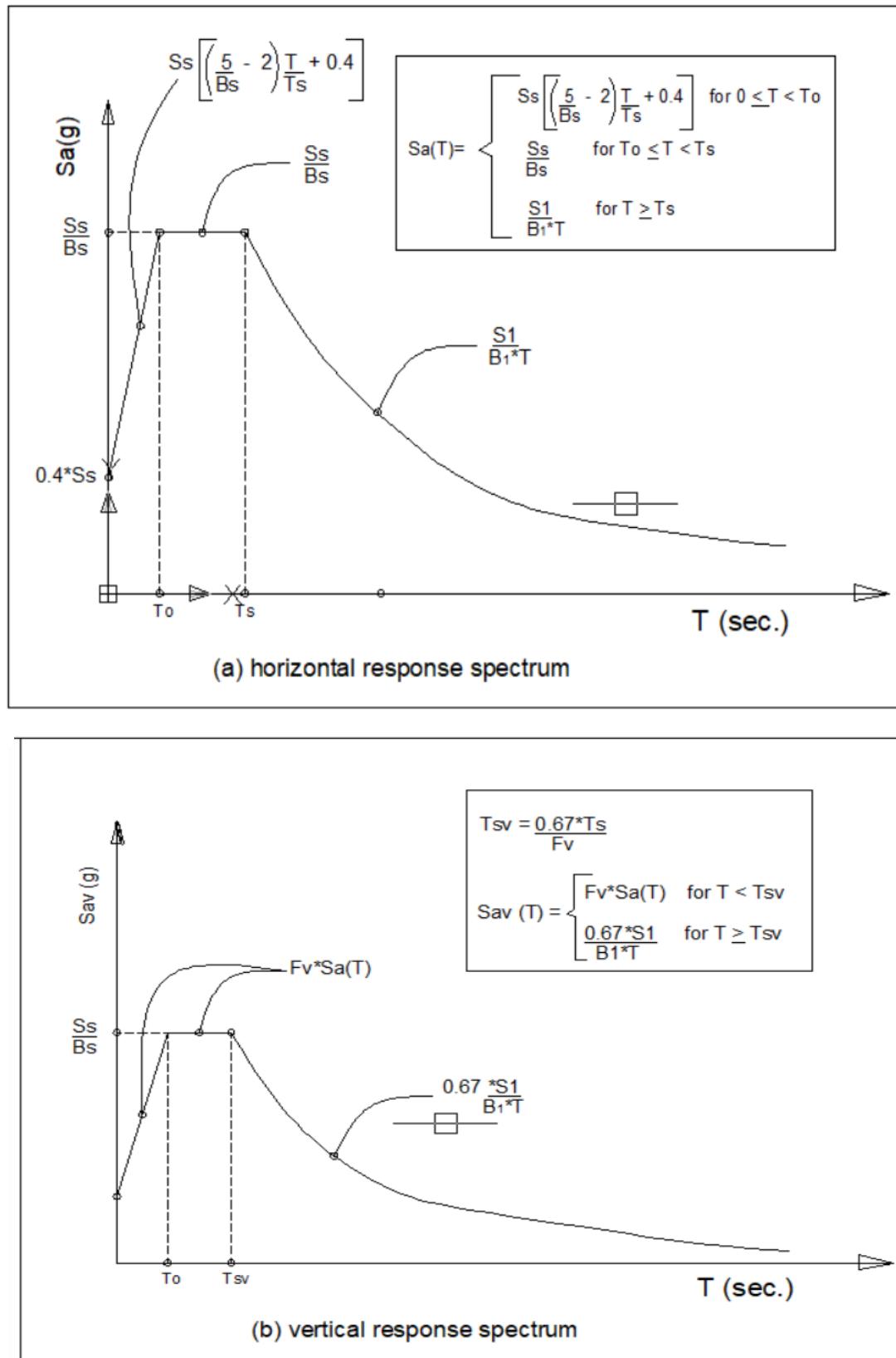


Figure 2-3b Standard response spectra (a) horizontal and (b) vertical (USACE, 2007)

(b2) Standard Elastic Response Spectra: Ethiopian code: ES EN 1998-1:2015

According to ES EN 1998: 2015 “Design of Structures for Earthquake Resistance. Part-1: General Rules – Seismic Actions and Rules for Buildings,” the Ethiopia territories are divided into seismic zones of constant hazard level, described in terms of the reference peak ground acceleration (PGA) on rock site (Type A ground condition), corresponding to the return period for no-collapse requirement, T_R , with probability of exceedance, P_E , in the structure’s life time of 50 years.

Special structures like large dams are beyond the scope of this code. However, spectral shapes are mostly similar for hazard levels with various return periods and probabilities of exceedance for a given national territory or particular region. Hence, the Ethiopian code provision may be referred to, in the absence of other applicable guidance, for comparison with the USACE (2003) guidance and for use in the development of standard response spectra that will be used as reference spectra for developing candidate acceleration time histories.

In this code, the Type-2 elastic response spectra ($M_s > 5.5$) are defined by considering the following criteria/parameters:

(i) Ground classification

Table 2-11 Ground classification

Ground class	Description	Shear wave velocity, V_s (m/s)	Reference
A	Rock or other rock-like geological formation	> 800	ES EN 1998-1:2015
B	Deposits of very dense sand, gravel, or very stiff clay	360 - 800	

(ii) Parameters defining horizontal elastic response spectrum

Table 2-12 Parameters defining standard horizontal elastic response spectra

Ground class	S	T_b (s)	T_c (s)	T_d (s)	Reference
A	1.0	0.15	0.4	2.0	ES EN 1998-1:2015
B	1.2	0.15	0.5	2.0	

(iii) Parameters defining the vertical elastic response spectra

Table 2-13 Parameters defining standard vertical elastic response spectra

a_{vg} / a_g	T_b (s)	T_c (s)	T_d (s)	Reference
0.45	0.05	0.15	1.0	ES EN 1998-1:2015

(iv) **Schematic of the standard horizontal and vertical response spectra** (Figure 2-4 (a) and (b)).

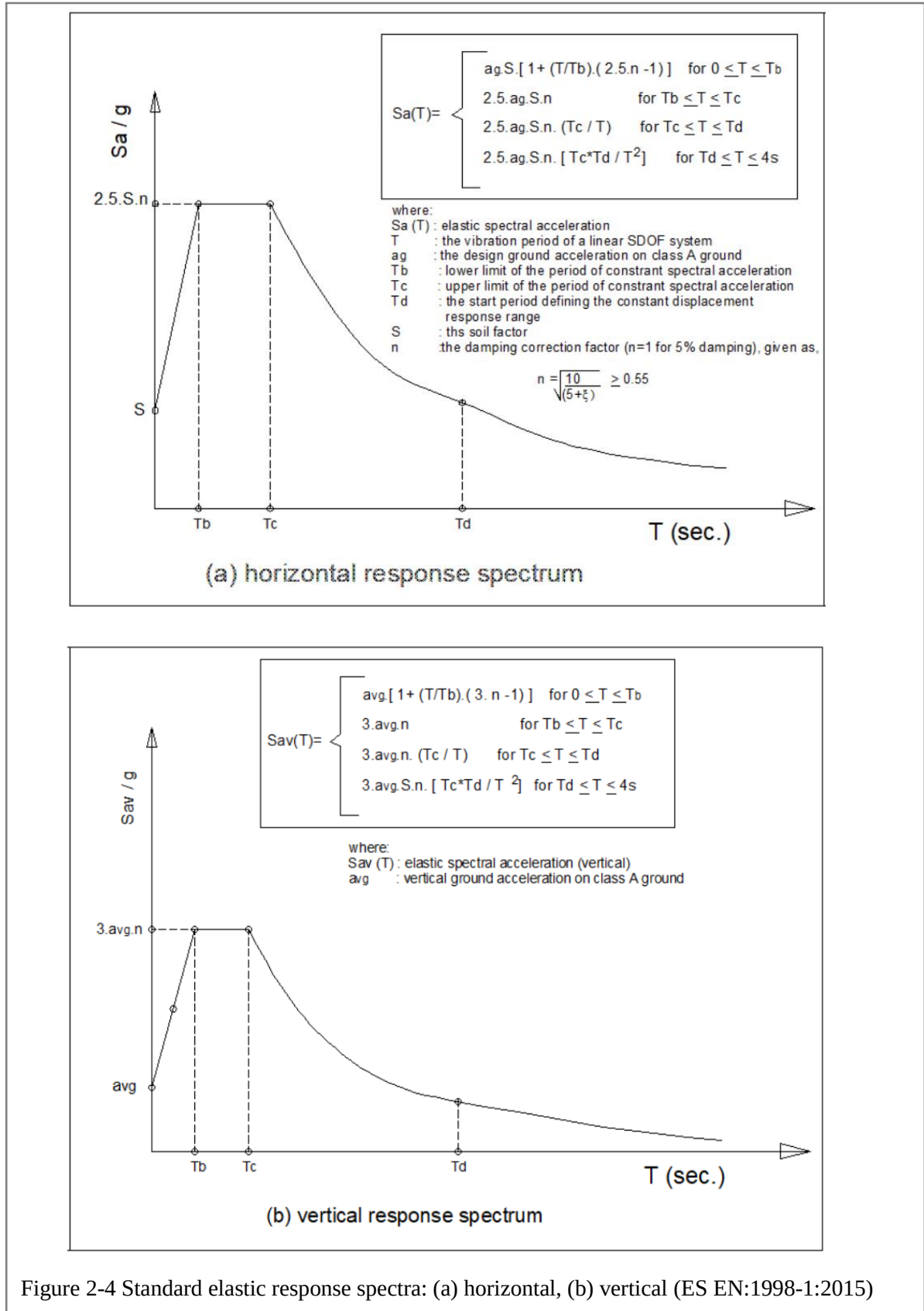


Figure 2-4 Standard elastic response spectra: (a) horizontal, (b) vertical (ES EN:1998-1:2015)

2.4.3.4 Site-Specific Acceleration-Time Histories

Site-specific acceleration time-histories of ground motion for seismic/dynamic analysis of concrete gravity dam are developed with the purpose to have a set (or sets) of time-histories that are appropriate for the types of analyses planned for particular structures and that are representative of site ground motions for the design earthquakes (USACE 2003, EM 1110-2-6051).

As recommended by USACE (2003), EM 1110-2-6051, the general approach for developing acceleration time-histories for the seismic analysis/performance evaluation follows the below steps ,

Step 1: Initial Selection of Suitable Candidates for Acceleration-Time Histories

The initial selection of candidate time-histories will be carried out after the project's design earthquakes and ground motion characteristics have been determined. In general, the candidate time-histories should be reasonably consistent with the site's tectonic environment, design earthquake (magnitude, source-to-site distance, type of faulting), local site conditions, and design ground motion characteristics (response spectral content, duration of strong shaking, and special characteristics, such as near-source characteristics). If there aren't enough recorded motions, ground motion modeling techniques can be used to create simulated recorded time-histories. The smooth design response spectra, which also include the zero-period acceleration that is equal to the peak ground acceleration (PGA), duration of intense shaking, and particular ground motion characteristics (such as near-source pulsive motions) are the typical design ground motion characteristics. These design earthquake and ground motion characteristics are then used to pick time-history records from the database of recorded ground motions that are sufficiently consistent with the design parameters and conditions (USACE 2003, EM 1110-2-6051).

(a) Factors to be considered while selecting candidate time-histories:

The acceleration time-histories should be chosen, whenever possible, to be comparable to the design earthquake in the following ways:

- Tectonic environment;
- Earthquake magnitude, fault rupture mechanism (faulting type);
- Distance from the epicenter of an earthquake.
- Site and subsurface conditions: rock, soft or firm soil.
- Design response spectra characteristics: similarity in magnitude
- Duration of strong ground shaking < 1.5 times the duration specified for the design earthquake
- Sequencing and pulse characteristics of the fault rupture process (< 10 km).

It is frequently essential to modify the already-existing records or develop synthetic records that satisfy the majority of these requirements because it is not always possible to discover records that fit all of these characteristics. At least three separate earthquakes must be taken into account for the SEE ground motion during the safety check of a dam (Wieland, 2017).

(b) Recognized databases of representative peak ground motions for time-history analysis:

Among the well-recognized database for strong ground motions include the Pacific Earthquake Engineering Research Center (PEER) NGA-West2 database and the Consortium of Organizations for Strong- Motion Observation Systems (COSMOS).

The PEER NGA-West2 database (<https://ngawest2.berkeley.edu>) includes a very large set of ground motions recorded worldwide of shallow crustal earthquakes in active tectonic regimes. The database has one of the most comprehensive sets of metadata, including different distance measures, various site characterizations, and earthquake source data. The PEER Ground Motion database has proved to be very popular among engineers in the earthquake-related discipline, who are increasingly using it for selection and modification of records to analyze computer models of various infrastructure projects including large dams.

New Search

Load Sample Input Values Clear Input Values

Search

These characteristics are defined in the NGA-West2 Flatfile. You need to re-run Search when any of these parameters are updated.

Record Characteristics:

RSN(s) : RSN1,...RSNn

Event Name :

Station Name :

Search Parameters:

Fault Type :

Magnitude : min,max

RJB(km) : min,max

Rrup(km) : min,max

Vs30(m/s) : min,max

DS-95(sec) : min,max

Pulse :

Additional Characteristics:

Max No. Records : (<=100)

Suite

Spectral Ordinate :

Damping Ratio :

Suite Average :

Controls

Search Records

Figure 2-5 The PGMD NGA-West2 search engine window

Table 2-14 Selection criteria for searching candidate time-histories: PEER NGA-West2

Parameter	Description
Magnitude	This criteria requires providing possible range of moment magnitude, writing in the format of [min, max] or leaving as blank for no restriction
Fault Type	The different types of fault mechanisms included under this criteria include: (1) All types of faults; (2) Strike-Slip type (SS); (3) Normal /Oblique type ; (4) Reverse/Oblique type ; (5) SS+Normal; (6) SS+Reverse; (7) Normal+Reverse
D5-95(sec)	This is the duration defined as the time needed to build up between 5 and 95 percent of the total energy released during earthquake. This criteria restricts range of the significant duration of the records. The input should be written in the format of [min, max], or if no restriction is required to be specified, we may leave it as blank.
R_JB (km)	The Joyner-Boore distance is defined as the shortest horizontal distance from a site to the surface projection of the rupture plane. The criteria requires restriction on the range of Joyner-Boore distance, with the input in the format of [min, max], or we may leave as blank if no restriction is needed.
R_rup (km)	R_rup denotes the the closest distance to the rupture surface. Input should be given in range and in the format of [min, max], or we may leave as blank for no restriction
Vs30 (m/s)	Vs30 denotes the average shear wave velocity of the upper 30-meter subsurface profile which is an important seismic parameter to determine the dynamic behavior of soils or rocks. Input in the format of [min, max],
Pulse	This is to provide restriction on the pulse characteristics of the searched earthquake records. The options available are: (1) Any record; (2) Only pulse-like record; (3) No pulse-like record

Step 2: Modification of time-history records: Simple Scaling and Spectral Matching

While the time-histories are initially selected to be roughly consistent with the seismic setting, design earthquake, and design ground movements, these time-histories and their response spectra may nevertheless significantly deviate from the design ground motion levels in amplitude. This is due to the fact that design ground motion characteristics are frequently determined via statistical analysis of ground motion data. It is therefore necessary that the selected time-histories are scaled to ensure that they recorded data are compatible with the design response spectra that would be used to develop the final set(s) of time-history records to be used in dynamic analysis. The chosen time-histories may be modified or adjusted using one of two methods to bring them closer to the intended ground motion conditions.

(a) Simple Scaling:

In this method, the given time-history is modified by simple scaling (i.e multiplication by a single constant factor) that places its spectrum roughly at the same level as the design response spectrum over the period range that is relevant to the structural response. The scale factor will adjust the peaks and valleys of the spectrum of any recorded time-history in such a way that the scaling will produce a

spectrum that, at some periods, is higher than the smooth design spectrum and, at other periods, is lower. The selection and scaling of further time-histories will likely be required in order for the spectra of the collection of time-histories to aggregately match or suit the design spectrum.

USACE (20003) EM 1110-2-6051 provides the following guidance and considerations when using simple scaling approach:

- Each selected time-history is scaled by a single factor: great variance effect
- Minimum number of time-histories:
 - For linear dynamic analysis: atleast 3 (for each component of motion) for each design earthquake;
- Degree of spectrum fit of time-histories used with the design spectrum.
 - Spectrum fit for individual time-histories.
 - Aggregate spectrum fit for time-history sets: mean spectrum should not be more than 15% lower than the design response spectrum
- Considerations for dynamic analysis for multiple components of motion.
 - Use same scaling factors for each component,
 - Using different scaling factors for each component
- Advantages:
 - Time-histories are truly “natural” and represent time-domain and spectral characteristics of actual recordings (or simulated recordings),
- Disadvantage:
 - Some of the “natural” quality is lost if different components of the same record are scaled by different factors

As a general rule, time-histories selected for seismic analysis should not require scaling upward or downward by more than a factor of about 2 to bring their spectra to the approximate level of the design spectrum in the period range of significance for structural response.

(b) Spectral Matching

The amplitude of a time-history is first scaled, and then its frequency content is adjusted so that its spectrum matches the smooth design spectrum. Because the amplitude and frequency are both forcefully manipulated or changed, the ground motion no longer remains a "natural" ground motion. This procedure necessitates the use of computer software.

- Minimum number of time-histories:
 - For linear dynamic analysis, a single time history is sufficient for each component of motion for each design earthquake.

- Degree of spectrum fit of time-histories used with the design spectrum.
 - It is necessary to compute a mean spectrum from each unique spectrum of the time histories. In the period range of significance to structural responses, the mean spectrum should not be more than 15% lower than the design response spectrum
- Considerations for dynamic analysis for multiple components of motion
 - When two or three motion components are required, each one must closely fit the design horizontal and vertical spectrum.
- Advantage:
 - Fewer time-histories are required, at least for linear analysis.
 - The method does not greatly change the energy of a time-history (fairly small change on the time-domain characteristics), if the spectrum of the scaled time histories does not differ too much from the design spectrum
- Disadvantage:
 - The time-domain character of the record is altered to some degree

(c) Computer programs for scaling or modifying recorded accelerograms

A number of computer programmes have been developed to modify a recorded accelerogram so that the response spectrum of the resulting accelerogram roughly matches or fits the design spectrum. It is preferable to utilise procedures that modify a recorded accelerogram, rather than develop artificial accelerograms, since the recorded motion will contain time-domain features that are representative of actual ground motions. PEER online scaling, SeismoSelect, Seismomatch, and Seismograph are examples of such software applications.

- (i) SeismoSelect is a quick and easy way to search, select, scale, and download ground motion data from various online strong motion databases. The scaling approach is that the ground motion records and their spectra are multiplied by a constant scaling factor ([https:// seismosoft.com/products/seismoselect/](https://seismosoft.com/products/seismoselect/)).
- (ii) SeismoMatch is an earthquake response matching program that is conducted by adding (subtracting) elementary wavelets to (from) the initial earthquake accelerograms so as to fit a particular target response spectrum. The program uses either the wavelets algorithm described by Al Atik and Abrahamson [2010] or the algorithm proposed by Abrahamson [1992] and Hancock et al.(2006). In this approach, once the a few accelerograms are loaded and the target response spectrum is set, users only need to specify the spectral matching algorithm, change the spectral matching period range, establish appropriate oe necessary tolerance, and the spectral matching is then executed. Convergence with the design spectrum was confirmed by checking that the spectral values of the modified time-history are within specified deviations from the design values at every

period being matched. SeismoMatch also includes a module that allows users to combine multiple matched accelerograms to generate a combined mean matched spectrum that meets the user's maximum and mean misfit requirements (<https://seismosoft.com/products/seismomatch/>).

Step 3: Further modification of time-histories to account for local conditions

Site response analyses may be carried out in which the rock time-histories are propagated through an analytical model of the site soil profile if time-histories for the bedrock beneath a site have been generated and it is intended to modify them for the overlaying soil profile. Since it is typically believed that the acceleration time-histories depict ground motions on level ground, a study to account for topographic impacts at a site with highly variable topography could also be taken into consideration.

2.4.3.5 Multi-Directional Effects of Earthquake Ground Motion

Structures must be capable of resisting maximum earthquake ground motions occurring in any direction. A straight gravity dams that are usually idealized as 2-D structures are generally analyzed for one or two components of the earthquake ground motion (horizontal only, or horizontal + vertical).

To get the maximum response from a time history analysis, it can be essential to apply horizontal components in either horizontal direction. The primary axes of the structure are frequently used to apply the orthogonal components of earthquake ground motion. By adding the individual reactions to each component of the ground motion, the overall response resulting from all of its components is calculated algebraically. Numerous studies show that, the critical direction of the horizontal ground motion components generating the greatest structural response relies on the two horizontal spectra as well as the structural response parameters, without regard to the vertical spectrum. However, a conservative technique is to assess the structure using the same horizontal spectrum and vertical spectrum applied concurrently along the two horizontal structural reference lines (USACE, EM 1110-2-6053, 2007). The USACE guideline (2007) suggests that multi-directional effects be considered when constructing or evaluating concrete hydraulic structures for seismic ground vibrations, according to the guidelines presented in Table 2-15 below, in which for two-component excitation a total of four permutations will be required.

Table 2-15 Earthquake load combination cases for combining static and dynamic stresses for 2-D analysis

Load Case	Seismic loads		Static Loads (stress)	Reference
	Horizontal (H)	Vertical (V)		
1	1	1	1	USACE (2007)
2		1	1	
3	1		1	
4			1	

Note: The number '1' denotes the loads are to be included in the analysis to account for the most unfavorable earthquake direction. For example, Case-1 = H + V + Static loads

2.5 Loads Acting on Concrete Gravity Dams

The stability, stress and deformation analysis of concrete dams require the computation of the loads acting on the dams. Concrete gravity dams may be subjected to the following loads in general, which can be considered in the design of new dams and/or the evaluation of existing dams (USACE 1995, EM 1110-2-2200).

(a) Primary loads: those loads of major importance to all dams irrespective of type and size

- Self-weight (dead load) of the dam.
- Water pressure/loads: headwater and tailwater pressures
- Uplift
- Earth pressure
- Reaction of foundation.

(b) Secondary loads: those loads that are universally applicable although of a lesser magnitude or alternatively are of major importance to certain types of dams.

- Silt pressures
- Temperature
- Wind pressure
- Wave pressure
- Ice pressure.
- Subatmospheric pressure (for overflow spillway sections only)

(c) Exceptional loads: loads considered in extreme events on the basis of limited general applicability of occurrence

- Earthquake loads

The schematic in Figure 2-6 below shows the primary loads acting on a typical concrete gravity dam.

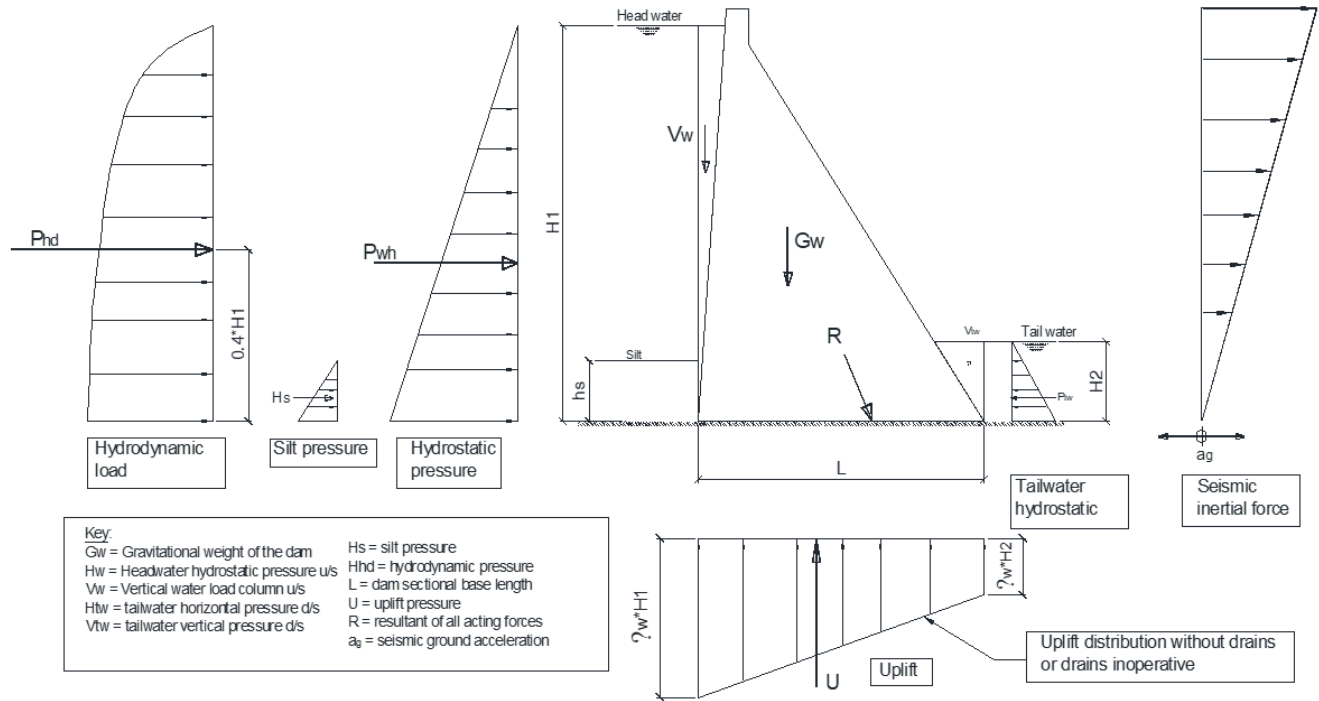


Figure 2-6 Typical schematic of forces acting on non-overflow concrete gravity dam section

Depending on how they act on the dam, the above loads can be represented as static, hydrodynamic, or seismic loads for structural analysis. For ease of 2-D structural analysis, the loads are computed per metre length of dam for a two-dimensional transverse section with unit width parallel to the dam axis.

2.5.1 Gravity Loads

Gravity loads are the principal stabilizing factors that comprise the weight of the dam plus any other ancillary structures such as gates, piers, and bridges. For computational reasons, the dead load is computed as the cross sectional area of the dam section under consideration multiplied by the unit weight, and it acts at the dam's sectional component's center of gravity. Unit weights can be calculated using relevant tests on concrete properties or, if no test data is available, from published sources. Normally, relatively small voids such as galleries and drainage pipelines are neglected from the dead load calculation unless the designer determines that the voids form a considerable fraction of the dam's volume and have an appreciable effect on the structure's stability.

$$W = \sum \gamma_i * V_i \quad (2.11)$$

Where: W = weight per unit length of the structure (KN/m)

γ_i = unit (or specific) weight of the material of the i^{th} structure component (kN/m³);

V_i = volume per linear meter length of the i^{th} structure component (m³/m).

The total weight of the dam acts at the center of gravity of the dam cross-sectional area.

2.5.2 Reservoir Loads

Using pressure loads or fluid elements, reservoir load is applied to the dam's upstream face and the reservoir bottom. This pressure should also be supplied to the downstream face and bottom of the tailrace region in circumstances of high tailwater (USBR, 2006).

(a) Reservoir (headwater) pressure

The hydrostatic pressure acting on the dam is determined by the water depth multiplied by the unit weight of water. Even though the weight changes significantly with temperature, the density of an incompressible water is 981 Kg/m³. The jet of water on an overflow section may exert pressure on the structure in some instances; however, in most cases, such pressures should be ignored in the stability analysis. The hydrostatic pressure always acts normal to the surface, and its intensity varies linearly with the depth of water measured below the free surface, as shown by the equation below.

$$p_{wh} = \gamma_w * h \quad (2.12)$$

Where: p = hydrostatic pressure at depth z

γ_w = specific weight of water 9.81 (kN/m³)

h = depth of water measured from water surface

For vertical upstream face, the hydrostatic pressure acts normal to the surface of the dam.

$$P_{wh} = 1/2 * \gamma_w * H^2 \quad (2.13)$$

where: P_h = total hydrostatic pressure load per unit length of dam (KN/m)

H = height of headwater (m)

The resultant hydrostatic force, P_{wh} , acts horizontally at a height of $H/3$ above the base of the dam. In the case of an inclined upstream face of dam, in addition to the horizontal hydrostatic pressure, the weight of the water column above the inclined portion of the upstream face is taken into account when calculating the total hydrostatic pressure, and this force acts through the centroid of the area representing the water column. Additionally, reservoir load is also applied to the reservoir bottom utilizing pressure loads or fluid elements (USBR, 2006).

(b) Tailwater Pressure

The tailwater loads are computed using both horizontal and vertical components acting on the sloped downstream face of the dam. These forces are normally computed using the full depth of the water for the nonoverflow sections of the dam; however, the flow conditions over the spillway and the depth of the backwater affect the tailwater loads for the overflow (spillway) section.

- (i) When the depth of the backwater is significantly more than the depth of the overflow, it is possible to compute the tailwater forces using all or almost all of the depth of the tailwater (USACE 1995, EM 1110-2-2200);
- (ii) When there is a sizable spillway discharge, the flow conditions cause a sizable hydraulic jump in the downstream channel, which can induce complex pressure distributions and lower the effective back-pressure against the dam. In such situations, according to past studies, tailwater retrogression can cause the effective depth of tailwater used to compute pressures and forces to be as little as 60% of the total tailwater depth (USACE, EM 1110-2-2200, 1995).

In cases of high tailwater, the tailwater pressure should also be applied to the downstream face of the dam as well as to bottom of the tailrace area (USBR, 2006).

2.5.3 Uplift Pressure and Drain Effectiveness: USACE (1995) Guidance

Uplift pressure is an internal water pressure that develops from headwater and tailwater and is present across horizontal cross sections inside the dam, at the dam-foundation interface, and within the foundation beneath the base. The water pressure exists within the cracks, pores, joints, and seams in the concrete and foundation material (USACE 1995, EM 1110-2-2200). By lowering the frictional (shear) resistance along probable slide lines, uplift pressures have an impact on the stability of gravity dams. Water pressures are normally exerted normal to the face of any surface exposed to water (USBR, 2006).

Uplift pressures vary over time and are influenced by the permeability of the material and boundary conditions. Earthquake loads are considered to have no effect on uplift pressures. The uplift pressure along the base of the dam will be regarded as acting over 100% of the base (USACE 1995, EM 1110-2-2200) with the effects of drains in operating or inoperative conditions as explained below in (a) and (b).

(a) Uplift Distribution: Without Drains (drains inoperative)

In case there are no drains or existing drains are inoperative within the dam, the uplift pressure is typically assumed to be a linear distribution of pressure from full headwater at the heel (upstream end of the dam cross-section) to full tailwater at the toe (downstream end of the dam). The hydraulic gradient will be assumed to vary in a straight line from headwater at the heel to zero or tailwater at the toe if no preparations have been made for uplift reduction. See Figure 2-7.

(b) Uplift Distribution: With Drains

Installing foundation drains helps lower uplift forces at or below the foundation. The drainage system's efficiency- which in turn depends on the depth, size, and spacing of the drains, the foundation's permeability, and the ease with which the drains may be maintained - determines the effectiveness of the reduction. The effectiveness is assumed to vary between 25% and 50% (see Figure 2-8).

Figure 2-7 and Figure 2-8, respectively, show the USACE (1995) guidance on computing uplift for dams without drains (drains inoperative) and those with drainage galleries (drains in operating condition).

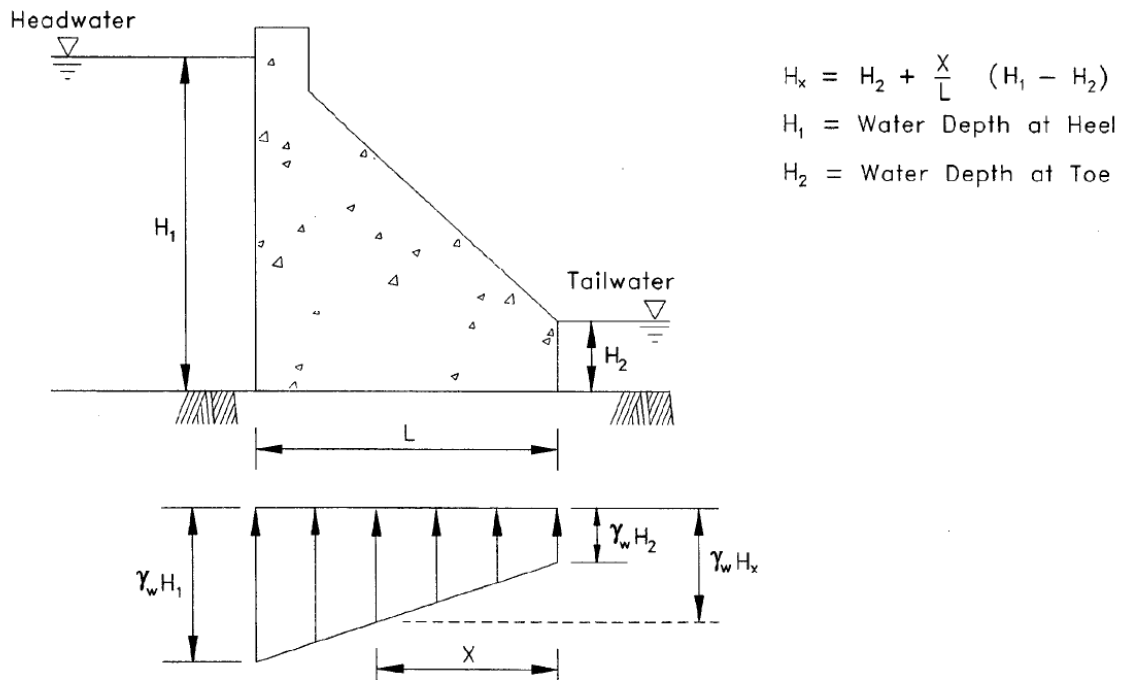


Figure 2-7 Uplift pressure distribution without drains (USACE 1995)

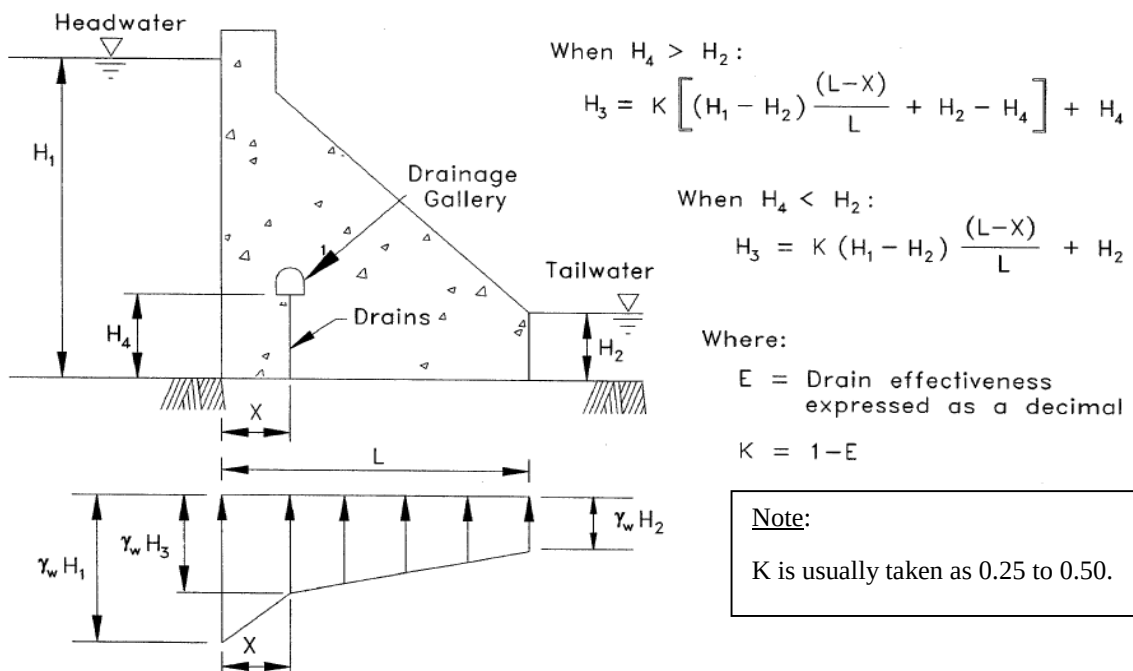
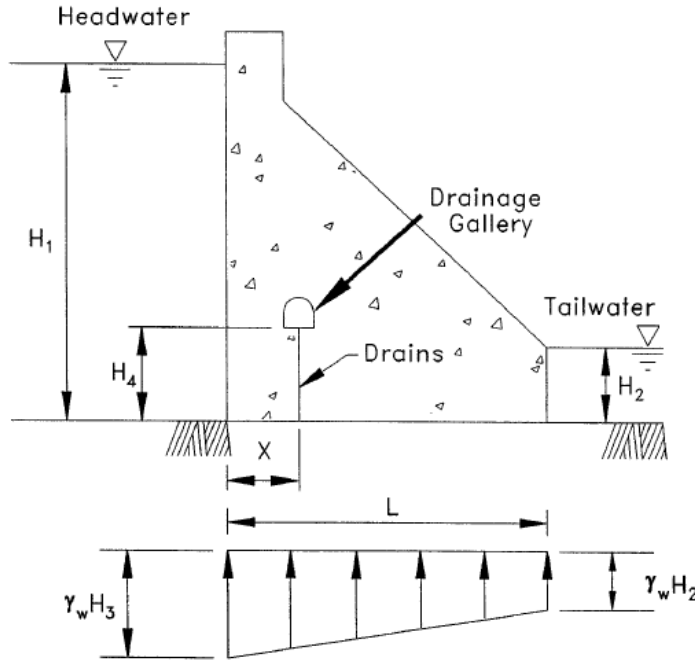


Figure 2-8 Uplift pressure distribution with drainage gallery (USACE, 1995)



Where:

E = Drain effectiveness expressed as a decimal

$$K = 1 - E$$

When $x \leq 0.05H_1$

If $H_4 > H_2$:

$$H_3 = K(H_1 - H_4) + H_4$$

If $H_4 < H_2$:

$$H_3 = K(H_1 - H_2) + H_2$$

Figure 2-9 Uplift pressure distribution with foundation drains closer to upstream face (USACE, 1995)

2.5.4 Earth Pressure

Earth pressures will be considered if sediment accumulates or soil backfill is placed against the face of the concrete dam. The fill material may or may not be submerged. The lateral earth pressures may be in an active or an at-rest state depending on the resulting structure lateral deformation. The earth pressure is calculated by:

$$p_{eh} = \gamma_s \cdot h_e \cdot k \quad (2.14)$$

where: p_{eh} = horizontal silt pressure per unit length of dam (KN/m²)

h_e = depth of earth fill (m)

k = coefficient of lateral earth pressure (at-rest or active, as appropriate)

The earth pressure acts horizontally at a height of $h_e/3$ above the base of the dam.

2.5.5 Silt Loads

When estimating silt loads, the effects of the water within the silt are taken into account along with the horizontal and vertical loads that can be assumed. Traditionally, the silt load on the dam is calculated using a 100-year sedimentation level. However, the service life of any future dams will need to be taken into account, as well as whether a higher silt load would be essential to the stability of the structure (USBR, 2017).

Silt pressure is calculated by the following equation:

$$p_{sh} = \gamma_s \cdot h_s \cdot k_o \quad (2.15)$$

where: p_{sh} = horizontal silt pressure per unit length of dam (KN/m²)

γ_s = submerged unit weight of silt (KN/m³)

h_s = depth of silt above bed level (m)

k_o = coefficient of at-rest lateral earth pressure

The horizontal component of silt pressure acts horizontally at a height of $h_s/3$ above the base of the dam, and the vertical component acts at the centroid of the cross-sectional area of the silt.

2.5.6 Temperature Loads

Temperature loads are those loads that are imposed to the concrete dam when concrete experiences a temperature change that causes a volumetric change (USBR, 2017). Temperature stresses need not be included in seismic load cases unless there are exceptionally harsh climatic conditions (USACE 1995, EP 1110-2-12).

Both CVC and RCC have generally similar thermal characteristics, which may be obtained from laboratory tests or from tests on concrete with similar aggregates from other sites. Temperature variations across the seasons have less of an impact on concrete gravity dams. Thermally induced stresses are minimized since gravity dams are free to expand upstream, downstream, and vertically (except at the dam-to-foundation contact). Due to the thickness of the dam, interior temperatures in gravity dams tend to stay fairly consistent, minimizing expansions and contractions (USBR, 2006).

2.5.7 Environmental Loads

As described in USACE 1995, EP 1110-2-12, there are various types of loads for which it is difficult or impossible to determine, with any degree of logic or accuracy, the exact magnitude and load pattern that would exist at the time of the design earthquake event. However, when compared to the dynamic and static loads previously stated, as well as the general character and range of magnitude often associated with loads of this sort, these loads typically do not significantly contribute to the results of the assessments for earthquake loading conditions. To ensure there are no unusual site conditions that might justify the inclusion of one or more of them in the seismic load combination cases, the designer should take a brief assessment and evaluation of these loads. These loads include pore pressures, thermal loads, wind loads, wave action, and ice loads.

(a) Wind Loads

Wind pressure acts on the exposed surface of the dam when wind blows. In this study, a few codes and guidelines regarding the consideration of wind loads in the design/evaluation of concrete gravity dams as described below. Below is brief description of some codes/guidelines regarding wind loads:

(i) USACE (1995) Guidance, EP 1110-2-12

According to USACE (1995), EP 1110-2-12, “Seismic Design Provisions for RCC Dams,” wind loads on concrete gravity dams are often negligible, making them unimportant for seismic design and evaluation. To be sure that there are no unusual site conditions that would justify including one or more of these loads in the earthquake load cases, the designer should at least conduct a reasonable evaluation of these loads based on the general nature and range of magnitude typically associated with loads of this type. Below are briefly discussed some guidelines on this subject.

(ii) Indian Standard: IS: 6512-1984 (Reprinted 1998)

Similar to the USACE (1995), the Indian Code IS:6512:1984 (Reprinted 1998) “Criteria for Design of Solid Gravity Dams” suggests that wind load exists as one of the environmental loads, but it is rarely taken into account when designing a dam, and thus, can be disregarded.

(iii) Compulsory Ethiopian Standard: CES 145 (2015)

The Compulsory Ethiopian Standard, CES 145 (2015): “Actions on Structures - Part 1-4: General Actions - Wind actions” is the latest version of the Ethiopian code that provides guidance on the determination of magnitudes and characteristics of natural wind actions for use in the structural design of buildings and civil engineering works based on the following basic concepts and principles

Basic Wind Velocity: The basic wind velocity, v_b , is defined as the basic wind velocity that is modified to account for the effect of terrain roughness and orography, and is calculated by the equation:

$$v_b = c_d \cdot c_s \cdot V_{b,0} \quad (2.16)$$

where:

v_b = the basic wind velocity, defined as a function of wind direction and season (time) of year at 10m above ground

$V_{b,0}$ = the fundamental value of the basic wind velocity, which defines the characteristic 10-minute mean wind velocity with an annual probability of exceedence of 2%, irrespective of wind direction and time /season/ of a year, at a height of 10 m above flat open terrain

c_d = the directional factor (recommended value is 1.0 as per the code)

c_s = the season factor (recommended value is 1.0 as per the code)

CES 145 (2015) recommends the use of constant basic wind velocity of 22 m/s as characteristic value having annual probabilities of exceedence of 2% that is equivalent to a mean return period of 50 years for all regions of Ethiopia.

Mean Wind Velocity: The mean wind velocity $[v_m]$ depends on the terrain roughness and the orography as well as on the basic wind velocity, v_b , and is calculated at a height 'z' above the terrain using the following formula.

$$v_m = c_r(z) \cdot c_o(z) \cdot v_b \quad (2.17)$$

where:

$c_r(z)$ = terrain roughness factor, that takes into account height above ground and ground roughness

$c_o(z)$ = orography factor, that takes into account the effects of hills, cliffs, etc. (usually assumed as 1.0 unless otherwise specified). The code also suggests that the effects of orography may be neglected when the average slope of the upwind terrain is less than 3 degrees.

Peak velocity pressure: The peak velocity pressure $q_p(z)$ at height z above ground, which includes mean and short-term velocity fluctuations, should be determined by the following equation.

$$q_p(z) = c_e(z) \cdot q_b \quad (2.18)$$

where:

ρ is the air density expected in the region during wind storms

$c_e(z)$ is the exposure factor, given by

$$c_e(z) = \frac{q_p(z)}{q_b} \quad (2.19)$$

q_b is the basic velocity pressure, given by

$$q_b = \frac{1}{2} \rho \cdot v_b^2 \quad (2.20)$$

where v_b = basic wind velocity

ρ = air density, which depends on the altitude, temperature and barometric pressure (recommended as 1.25 kg/m³)

If the effects of wind pressure variation with height of structure, terrain roughness, terrain orography, and wind turbulences are neglected, then the peak velocity pressure, $q_p(z)$, may then be taken equal to the basic wind velocity pressure, q_b , and is estimated using Eqs.2.21 below.

$$q_p(z) = q_b = \frac{1}{2} \rho \cdot v_b^2 \quad (2.21)$$

(iv) Reference basic wind velocity map for Ethiopia (Melesse, G., Melesse, H. & Devadass T., 2021)

In this research, reference basic wind velocity zone map for Ethiopia was developed by dividing Ethiopia into six zones based on wind velocity data collected from 159 stations across the country. According to this study, the basic wind velocity near or around Gibe III dam site can be estimated to be 30 – 35 m/s on the map

(b) Wave Action

Waves are generated on the surface of the reservoir by the blowing winds, which cause a pressure towards the downstream side of the dam. The transient hydrodynamic thrust caused by wave action against the dam's face is only taken into consideration in exceptional circumstances, according to Novak et al. (2007). Its influence is local and unpredictable by nature, and its magnitude is quite small in most cases. In situations where a typical wave pressure is required, a conservative approximation of the additional hydrostatic load at the reservoir surface is given by the empirical formula in Eqs.2.23 in page 60.

(i) USACE (1995) Guidance

USACE (1995) EM 1110-2-2200 suggests that, while wave pressures are of more importance in their effect upon gates and appurtenances, they may, in some instances, have an appreciable effect upon the dam. The height of waves, run-up, and wind setup are usually important factors in determining the required freeboard of any dam. Wave dimensions and forces depend on the extent of water surface or fetch, the wind velocity and duration, and other factors.

(ii) Indian Standard, IS:6512-1984 (Reprinted 1998)

In addition to the static water loads, the upper portions of concrete gravity dams are subject to the impact of waves. Wave pressure against massive dams of appreciable height is usually of little consequence. The force and dimensions of waves depend mainly on the extent and configuration of the water surface, the velocity of wind and the depth of reservoir water.

An empirical method developed by T. Saville is recommended for computing wave height as illustrated in the wave pressure diagram in Figure 2-10 below.

The maximum unit pressure p_{wmax} in kPa occurs at $0.125h_w$ above the still water level

$$p_{wmax} = 24 h_w \quad (2.22)$$

The total wave force P_w (in kN) is given by the area of the triangle a-b-c in Figure 2-10.

$$P_{wv} = 20h_w^2$$

where h_w = height of wave (m)

The center of application is at a height of $0.375 h$, above the still water level.

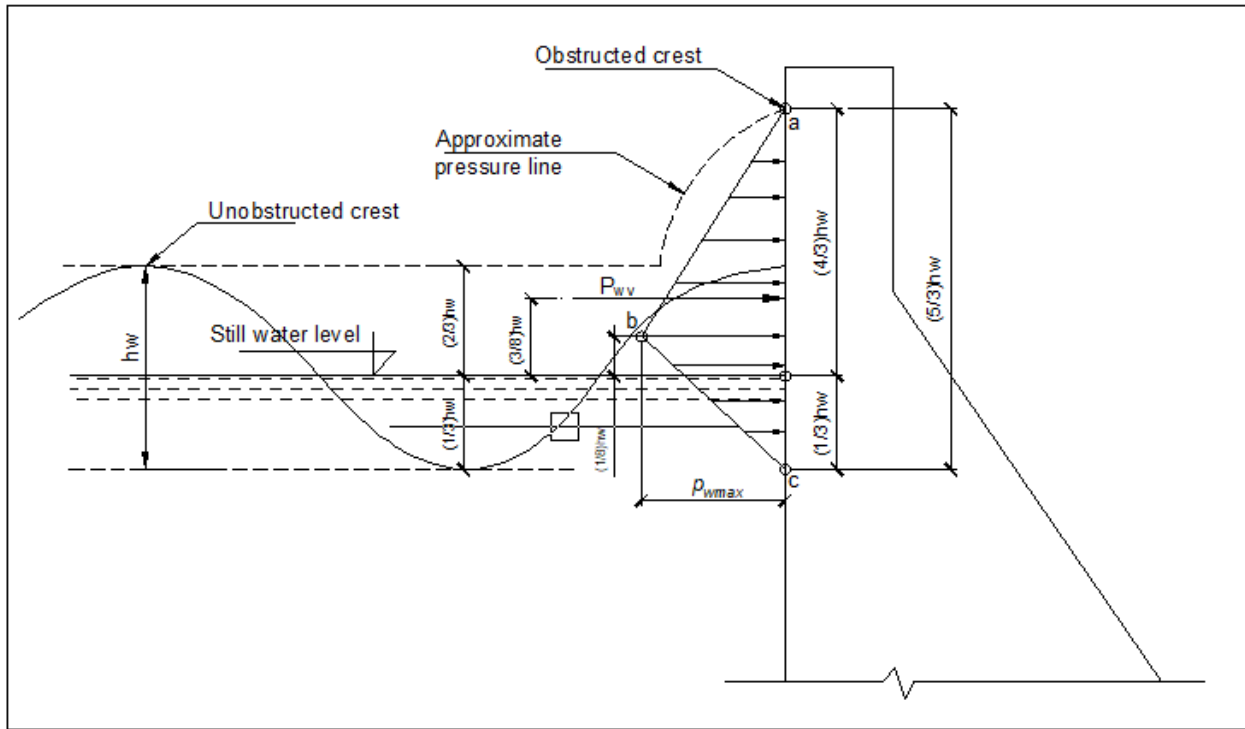


Figure 2-10 Wave pressure acting on concrete gravity dam (IS:6512-1984, Reprinted 1998)

(iii) Maximum Wave Pressure and Wave Height (Novak et al., 2007)

Wave pressure depends on the wave height (hw) which in turn depends on the fetch (F), which corresponds to the maximum free distance which wind can travel over the reservoir, and the wind velocity (U). The maximum additional hydrostatic force due to wave action is computed using the wave pressure pressure envelope similar to that shown in Figure 2-10 with the following empirical equation.

$$P_{wv} = 2\gamma_w h_w^2 \quad (2.23)$$

For short fetch lengths ($F \leq 32$ km), height of wave, hw , can be estimated by

$$hw = 0.032 (UF)^{1/2} + 0.763 - 0.271(F)^{1/4} \quad (2.24)$$

For large values of fetch ($F > 32$ km) the last two terms may be ignored.

$$hw = 0.032(UF)^{1/2} \quad (2.25)$$

where: hw = height of wave (m)

U = wind velocity (km/hr)

F = Fetch (km)

(c) Ice loads:

USACE 1998, EM 1110-2-12, suggests that ice loads need not be included in an earthquake load case unless there are extraordinary climatic conditions that would result in a significant depth of ice to exist for a prolonged length of time.

2.5.8 Earthquake Loads

The dam, foundation, and reservoir all move and interact with one another during an earthquake; as a result, the most realistic study would take into account all of these interactions (USBR, 2006).

According to ICOLD (2010), the controlling earthquake ground motions for seismic safety and performance evaluation of concrete gravity dams are Safety Evaluation Earthquake (SEE) for evaluating strength requirements and Operating Basis Earthquake (OBE) for serviceability requirements as discussed in section 2.6.1 and section 2.6.1 respectively. The hazard levels and the peak ground motions corresponding to these design earthquakes can be determined by conducting site-specific seismic hazard studies – DSHA or PSHA, as discussed in section 2.4.2.2 (a) and (b) respectively. The dynamic method of analysis determines the structural response using either a response spectrum or acceleration-time records for the dynamic inputs (USACE 1995, EM 1110-2-2200).

- In both RCC and CVC gravity dams, seismic loads are induced in similar manner. For calculating the seismic load response of small gravity dams built on rock foundations, a pseudo-static analysis is a suitable method. But in order to determine the internal stresses caused by earthquakes in medium and large dams, a state of the art dynamic study using FEM is needed (USBR, 2017).
- Earthquake loadings should be verified for horizontal earthquake acceleration as well as vertical earthquake acceleration if included in the stress analysis. While an earthquake might accelerate in any direction, the study should focus on the most unfavorable direction (USACE 1995, EM 1110-2-2200). During earthquakes, stresses in gravity dam structures are generated due to the inertia of their own mass and the hydrodynamic loads of the reservoir water operating on the dam (USBR, 2017).

In simple seismic-coefficient (pseudo-static analysis), the inertia of dam concrete structure for horizontal earthquake acceleration is defined by the force required to accelerate the dam's concrete mass, as given by the following equation:

$$P_{eh} = m \cdot a_x = (W/g)(\alpha_x) (g) = W \cdot \alpha_h \quad (2.26)$$

where

P_{eh} = horizontal earthquake force (N)

m = mass of dam (kg)

a_x = horizontal earthquake acceleration = $\alpha_h \cdot g$

W = weight of dam (N)

g = acceleration due to gravity (9.81m/s²)

α_h = seismic coefficient in horizontal direction

In addition to the inertia of the dam body, hydrodynamic loads are the forces that arise from inertia of the reservoir as a result of the interaction between the reservoir and the dam during an earthquake, and this is briefly discussed in section 2.5.9 below.

2.5.9 Hydrodynamic Loads

During an earthquake event, the dam and reservoir move and interact due to hydrodynamic pressures at the structure-water interface. Along with concrete inertia forces, hydrodynamic pressure causes an increase or reduction in pressure on the dam. This interaction can have a substantial impact on the dam's earthquake reaction (deformation and stresses) and must be incorporated in any dynamic analysis in which the complete formulation of the fluid-structure interaction results in hydrodynamic pressures that can be read as an additional force, mass, and damping (Chopra 1987, as cited in USACE 2003).

The hydrodynamic force may be computed by means of the following models:

- 1) Simplified added mass models:
 - The Westergaard's added mass theory (1933)
 - Zangar added mass model (1952)
 - The Generalized Westergaard 'added mass' theory (Kuo, 1982),
- 2) A more refined analysis using finite element formulation of the dam-reservoir-foundation system, in which the dam, reservoir and foundations are modelled separately with distinct boundary conditions at their interfaces.
 - The Eulerian approach
 - The Lagrangian approach

The simplified added mass concepts (no.1) and their limitations are briefly discussed below in sections 2.5.9.1 through to section 2.5.9.4. The refined methods using the Eulerian and Lagrangian approach utilizing the finite element formulation of the dam-reservoir interactions are out of the study scope and thus not dealt with herein.

2.5.9.1 The Westergaard's Added Mass Approximation

Westergaard (1933) developed an analytical solution of the hydrodynamic pressure distribution for a dam-reservoir interaction analysis for a concrete gravity dam with vertical upstream face, and this formula has been used in engineering practice in preliminary analysis and design of concrete dams to this day. The hydrodynamic forces exerted on a dam owing to ground motion, according to Westergaard, are similar to the inertia forces of a volume of water attached to the dam face. This volume of water is moving back and forth with the dam as the dam is shaken during earthquake while the remainder of the reservoir is dormant, The Westergaard's added mass model was based on the following assumptions:

- the dam is idealized as a 2-D rigid monolith system with vertical upstream face;
- the dam is considered rigid block, and thus does not deform. The natural period of free vibration of the dam is assumed to be significantly smaller than the period of earthquake excitation (no resonance)
- infinite reservoir in the upstream direction with constant depth;
- incompressible reservoir (water) particles;
- surface waves are ignored;
- only horizontal ground motion (sinusoidal oscillations) in the upstream-downstream direction considered.

The Westergaard's approximate parabolic function that describes the hydrodynamic pressure on the face of the dam at a depth, z , above the bottom end of the reservoir is given by Eq.2.27 below.

$$p_{hd} = \frac{7}{8} \cdot \alpha_h \cdot \rho_w \cdot \sqrt{H(H - z)} \quad (2.27)$$

where: p_{hd} = hydrodynamic pressure calculated at depth, z , above reservoir bed (N/m²)

ρ_w = mass density of water (Kg/m³)

α_h = coefficient of maximum horizontal acceleration of foundation = a_g / g

a_g = earthquake acceleration of ground (m/s²)

g = acceleration due to gravity ($g = 9.81$ m/s²)

z = depth of water measured from bottom of reservoir (m)

H = depth of reservoir water (m)

The assumption of equivalent added mass of water is often applicable in circumstances when the reservoir responses are at considerably higher frequencies than the structure's fundamental frequency. The notion of 'added mass' is defined in such a way that a mass of a certain body of water is attached to the dam face and forcefully moving with the dam during ground motion - a concept that has substantially simplified the dynamic response analysis of dams with hydrodynamic effects. Figure 2-11 shows a typical schematic of hydrodynamic loads on a seismically loaded concrete gravity dam (USACE 2003, EM 1110-2-6051).

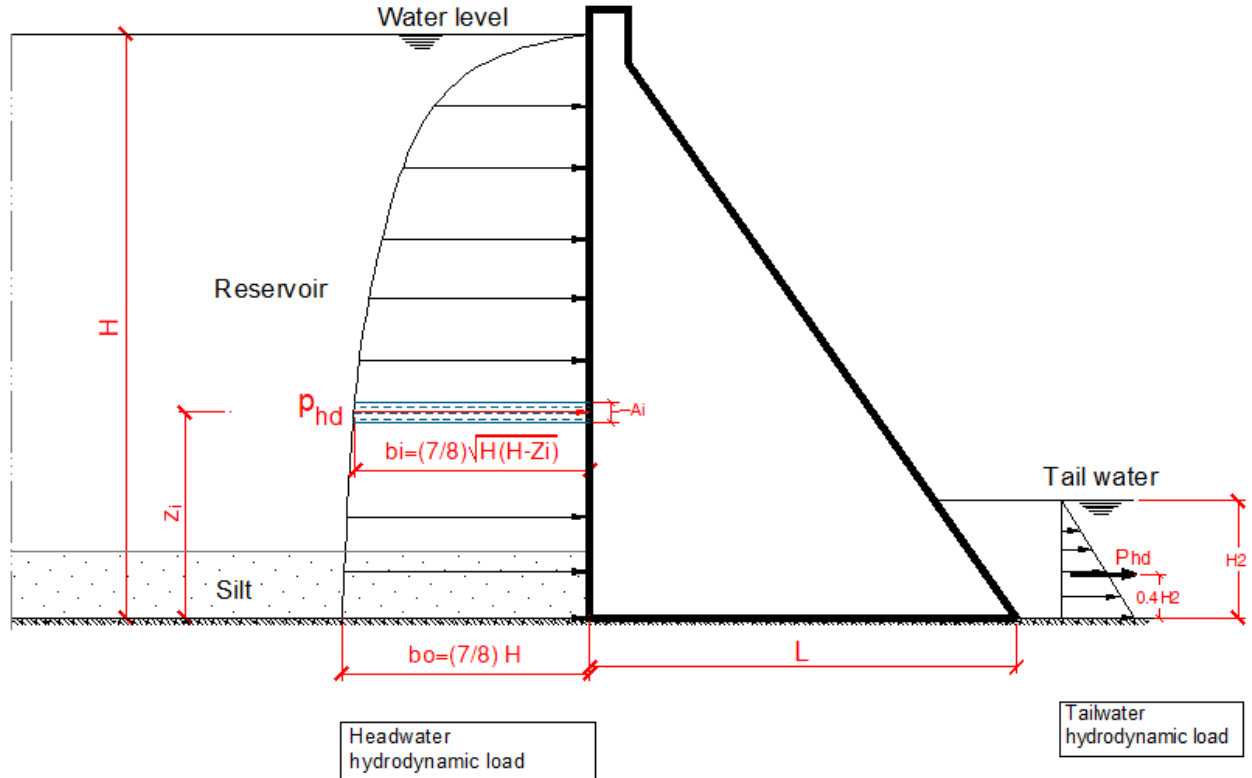


Figure 2-11 Westergaard hydrodynamic added-mass model

The hydrodynamic pressure exerted normally on the upstream face of the dam, at height z_i above the base of the dam, is equivalent to the inertia force of a prismatic body of water of unit cross-section (1m) and width, b_i , described by a parabola expressed by Eqs.2.28 below

$$b_i = \frac{7}{8} \sqrt{H(H - Z_i)} \quad (2.28)$$

where:

b_i = width of the volume of added mass of water per unit width at depth of interest, z_i , (m)

$b_o = 7/8 \cdot H$ = width of the volume of added mass of water per unit width at reservoir bottom, i.e. at $z_i=0$ (m)

The added mass of water, m_{ai} , at a given location, z_i , above the bottom of reservoir can be obtained by multiplying the mass density of water, ρ_w , by the volume of water tributary to point i :

$$m_{ai} = \frac{7}{8} \cdot \rho_w \cdot \sqrt{H(H - z)} A_i \quad (2.29)$$

where:

A_i = tributary surface area of added mass at point i

2.5.9.2 Zangar's Added Mass

By adjusting the Westergaard's added mass for water against a sloping surface, Zangar (1952) established a method for determining the rise in water pressure caused by a horizontal earthquake. Zangar's work comprised sloped surfaces at varying angles that allowed for a reduced volume of water mass. He developed the following relationship between added hydrodynamic mass and reservoir depth:

$$m_{ai} = C * \rho_w * H * A_i \quad (2.30)$$

where:

m_{ai} = added mass at node 'i'

C = a dimensionless pressure factor, and is a function of y/H and the angle of inclination of the upstream face to the vertical, Θ , as given by the following equation:

$$C = \frac{C_m}{2} \left[\frac{y}{H} \left(2 - \frac{y}{H} \right) + \sqrt{\frac{y}{H} \left(2 - \frac{y}{H} \right)} \right] \quad (2.31)$$

y = the depth of node i below water surface at which the pressure increase is being determined

H = the depth of reservoir at the section of dam considered for analysis (m)

ρ_w = mass density of water (kg/m³)

C_m = the maximum value of C for a specified upstream slope, given by:

$$C_m = 0.73 \frac{\theta}{90} \quad (2.32)$$

Θ = the angle of inclination of the upstream face to the horizontal (degrees)

A_i = tributary area associated with node 'i' (m²)

2.5.9.3 The Generalized Westergaard's Added Mass Model (Kuo, 1982)

While the application of the simpler Westergaard's added mass formula is restricted to planar vertical surfaces, Kuo, S.J. (1982) produced the Generalized Westergaard's formulation, which can be seen an extension of the original Westergaard formulation (1933). This approach assumes the same added mass concept for an incompressible fluid, which is still expressed by the original Westergaard's parabolic shape; however, the hydrodynamic pressure at any point 'i' on the upstream face of the dam is proportional to the total acceleration acting normal to the dam at that point 'i' with its magnitude equal to the inertia force produced by a prismatic body of water of unit cross-section area, A_i , and length at point, computed by Eqs.2.18. Because it only depends on the total normal acceleration at local points, the "added mass" is generalized to be relevant to the general geometry (sloped or curved contact surfaces) of the dam's upstream face.

Assuming that the impounded water is incompressible, the dam-water interaction effects can be represented by the finite element model of dam with incompressible water added as lumped mass on upstream dam face, and the hydrodynamic pressure acting normal to the upstream face of dam at a particular node 'i' is given by:

$$p_{ni} = \alpha_i \ddot{a}_{ni} \quad (2.33)$$

where:

p_{ni} = hydrodynamic pressure at node "i" on upstream face (compression as positive);

$\ddot{a}_{ni}$ = total normal acceleration at node 'i' on upstream dam face

α_i = Westergaard pressure coefficient, given by:

$$\alpha_i = \frac{7}{8} \cdot \rho_w \cdot \sqrt{H(H - Z_i)} \quad (2.34)$$

H_i = depth of water at vertical section that includes point i

Z_i = height of point i above the base of the dam

The numerical modeling / finite element formulation of the Generalized Westergaard added mass model (Kuo, 1982) is discussed in Section 2.10.1.1.

2.5.9.4 Limitations of The Simplified Added Mass Models

The simplified added mass models described in sections 2.5.9.1, 2.5.9.2, and 2.5.9.3 above have the following limitations or criticisms.

- (i) Although the simplified Westergaard's formula has a broad range of applications, it ignores the water-foundation interaction and water compressibility. During an earthquake, this method estimates the mass of water interacting with the dam. If the dam face is not vertical or if the dam and foundation are not rigid, the mistake increases. Additionally, its application should be restricted to estimating seismically induced hydrodynamic loads on concrete gravity dams (USBR, 2018).
- (ii) According to the USACE 2003, EM 1110-2-6051, both the classical Westergaard's approach (1933) and the Generalized Westergaard's formulation are based on the assumption that they are typically valid in situations where the fluid responses are at frequencies much higher than the fundamental frequency of the dam structure. Because the restrictions established by the original Westergaard analysis are not satisfied, there is no reasonable foundation for the assumption in the generalized Westergaard analysis (Ghanaat, Y., 1993). Their usage should be restricted to feasibility or preliminary studies as they only provide an approximate estimate of the hydrodynamic forces acting on the face of the dam.

- (iii) A better approach of analysing the dam-reservoir interaction effects is to idealize the impounded water as an assemblage of finite fluid elements using the finite element method, which permits a realistic treatment of the variable geometries.

2.5.10 Other Secondary Loads

The other secondary loads such as wave pressure, subatmospheric pressure, etc, are needed to be considered depending on the environment, type and specific requirements needed to be considered in the design of new and/or safety and performance evaluation of existing dams (USACE 1995, EM 1110-2-2200).

2.5.10.1 Finite Element Added Hydrodynamic Mass Model

The finite element formulation of the Generalized Westergaard's added mass model (Kuo, 1982) is discussed in section 2.10.1.1.

2.6 Load Combinations for Seismic Response Analysis

The primary, secondary, and exceptional loads may not all be acting on a dam at once. A concrete dam should be designed to withstand the most unfavorable combinations of loads that have a respectable likelihood of occurring simultaneously.

According to USACE 1995 EM 1110-2-2200, OBE and MDE are the earthquake ground motions that are used in the construction and assessment of concrete gravity dams. The earthquake loads should be combined with other types of loads that are anticipated to occur during normal operations. The ultimate strength and serviceability requirements for the design and assessment of CMC and RCC dams, as stated in the USACE 2007, EM 1110-2-6053, and Ghamaat (2014), can be established for the following loading combinations.

The ultimate strength and serviceability requirements for the design and assessment of CMC and RCC dams, as stated in the USACE 2007, EM 1110-2-6053, and recommended by Ghamaat (2014), can be established for the loading combinations discussed in sections 2.6.1 and 2.6.1 below. The loading cases represent the total demand for which the structure must be designed or evaluated under earthquake loadings. It is not advisable to consider combining loads (such as flood and earthquake) that have low probabilities of occurring at the same time.

2.6.1 Serviceability Loading Combination

This loading combination is an unusual loading condition used to determine the total earthquake demand on concrete gravity dams for Operating Basis Earthquake (OBE) conditions:

Earthquake Load Case-1: OBE ground motion / reservoir empty

This is an unusual earthquake loading condition that consists of the loads that exist after the dam construction completed but prior to the start of reservoir filling (i.e. empty reservoir condition) (USSACE 1995, EM 1110-2-2200)

The load combination may expressed in simple equation as,

$$Q_{SER} = Q_{DL} + Q_{OBE} \quad (2.35)$$

The load cases that are to be considered include only dead loads of dam structure and all attachments and/or accessories, and the OBE ground motion in upstream direction

Earthquake Load Case-2: OBE ground motion / reservoir full condition

This is another serviceability loading condition after the reservoir is fully loaded and can be expressed by the following simple equation

$$Q_{SER} = Q_{DL} + Q_{LL} + Q_{OBE} \quad (2.36)$$

where:

Q_{SER} = combined action of loads due to dead, live, and OBE loads for use in the evaluation of serviceability performance

Q_{OBE} = earthquake loading effect due to OBE ground motions including hydrodynamic pressure effects

The live load case that should be taken into account, Q_{LL2} , are those that are most likely to be present during the OBE seismic event and often consist of the following:

- Operating basis earthquake (OBE).
- Horizontal earthquake acceleration in downstream direction
- Usual pool elevation.
- Minimum tailwater.
- Uplift at pre-earthquake level.
- Silt pressure, if applicable.
- No ice pressure

2.6.2 Strength Design Loading Combination

This loading combination is an extreme loading condition used to determine the total static plus earthquake demand on plain concrete structures for Maximum Design Earthquake (MDE) conditions, which is equivalent to the SEE event that represents the maximum of MDE or MCE.

Earthquake Load Case-3: SEE ground motion / reservoir full condition

$$Q_{STR} = Q_{DL} + Q_{LL} + Q_{MDE} \text{ (or } Q_{SEE}) \quad (2.37)$$

Q_{STR} = Combined action of dead, live, and maximum design earthquake loads for use in evaluating structural strength or damage control performance

Q_{DL} = Dead load effect

Q_{LL} = Live load effect + uplift (The structure response to live loads including hydrostatic, ground pressure, silt, and temperature loads are included in the live load effects)

Q_{SEEMDE} = Earthquake load effect from SEE or MDE ground motions including hydrodynamic pressures

According to USACE 1995, EM 1110-2-2200, the following loading conditions are typically included in the computation of live loads that are likely to be present during earthquake event.

- Maximum credible earthquake (MCE) or Safety Evaluation Earthquake (SEE).
- Horizontal earthquake acceleration in downstream direction
- Usual pool elevation.
- Minimum tailwater.
- Uplift at pre-earthquake level.
- Silt pressure, if applicable.
- No ice pressure.

In order to account for the most unfavorable earthquake direction, the earthquake load may entail multiple ground motions, each of which is multiplied by +1 or -1.

Due to the difficulties in predicting the accompanying ground motion, the MCE is typically established using statistical analysis in engineering practices, with a typical return period of 10,000 years. As a result, the MDE or SEE are utilized in place of the MCE (Wieland, 2006). Thus, it is important to remember that some secondary loads, such as wind loads, thermal loads, and ice loads, need not be taken into account in earthquake response analyses of concrete gravity dams, as per USACE 1995, EM 1110-2-2200, due to their negligible effects in comparison to other primary loads of greater magnitudes and impacts on the structure.

2.7 Seismic Performance Criteria

2.7.1 ICOLD (2010) Approach

Concrete dams that are earthquake-resistant must be designed using the proper performance criteria, which must take into account both the intended level of safety and the specifics of the design and analysis processes. It is now possible to forecast with some accuracy the linear elastic response that would occur from a given earthquake input thanks to the development of contemporary computer analysis tools. The following performance parameters are listed by ICOLD (2010) for the main dam body/structure:

Table 2-16 Seismic performance criteria: ICOLD (2010) approach

Design earthquake event	Earthquake loading condition	Performance criteria
OBE	Unusual	No structural damage such as cracks, deformations, leakage, etc. which affect the operation of the dam and the reservoir, is permitted. Minor damages of repairable scale is accepted.
SEE	Extreme	Structural damage (cracks, deformations, leakage etc.) is accepted as long as the stability of the dam is ensured and no uncontrolled release of large quantities of water is released from the reservoir causing flooding in the downstream region of the dam.

2.7.2 USACE (2007) Approach

2.7.2.1 Seismic Performance Levels

As stated in USACE's 2007, Engineer Manual EM 1110-2-6053, several performance levels are taken into account when assessing how concrete gravity dams react to ground vibrations during earthquakes. Serviceability performance, damage control management, and collapse prevention performance are the most often employed performance levels.

(a) Serviceability performance

In the early aftermath of earthquakes causing ground vibrations up to the OBE level, the dam is anticipated to be operational and in good working order.

(b) Damage control performance

When the dam is subjected to severe seismic events, some elements of the dam may deform over their elastic limits (non-linear behavior) if non-linear displacement demands are low and load resistance is unaffected. Although damage may be substantial, it is typically confined to specific areas where cracking and joint opening take place. All potential damage areas should be noted by the designer, who should also be confident that the structure can withstand static loads and, if necessary, be repaired to prevent future damage from non-earthquake stresses.

(c) Collapse prevention performance

Regardless of the degree of damage, the dam must operate well in terms of collapse prevention. Although it shouldn't cause an uncontrolled flow of water, the dam may sustain irreparable damage from nonlinear deformation greater than those associated with damage control performance. If the dam is subjected to severe earthquake episodes without collapsing, it can be anticipated that its resistance will decline with increasing displacements. Checking this can help determine how stable

the environment will be after an earthquake. Only highly unlikely MCE or SEE events should be eligible for collapse prevention performance.

2.7.2.2 Seismic Performance Goals

Dams must be able to function under repeated, sustained loads. The serviceability requirement is to ensure that the dam will continue to operate without interruption and sustain little to no damage in the event of an unusual earthquake loading, or OBE. Dams made of plain concrete often exhibit limited ductile behavior in flexure and brittle behavior in shear. A restricted plastic range, which may include strain hardening or softening, and a range of elastic behavior are characteristics of limited ductile behavior (Figure 2-12). Finally, the strength is lost completely or partially, and it is at the residual level. Limited ductile structures should be strong enough to guarantee that performance will remain within the inelastic zone, where strength improves with an increase in strain (i.e. damage control region in Figure 2-12), under the MDE or SEE ground motions.

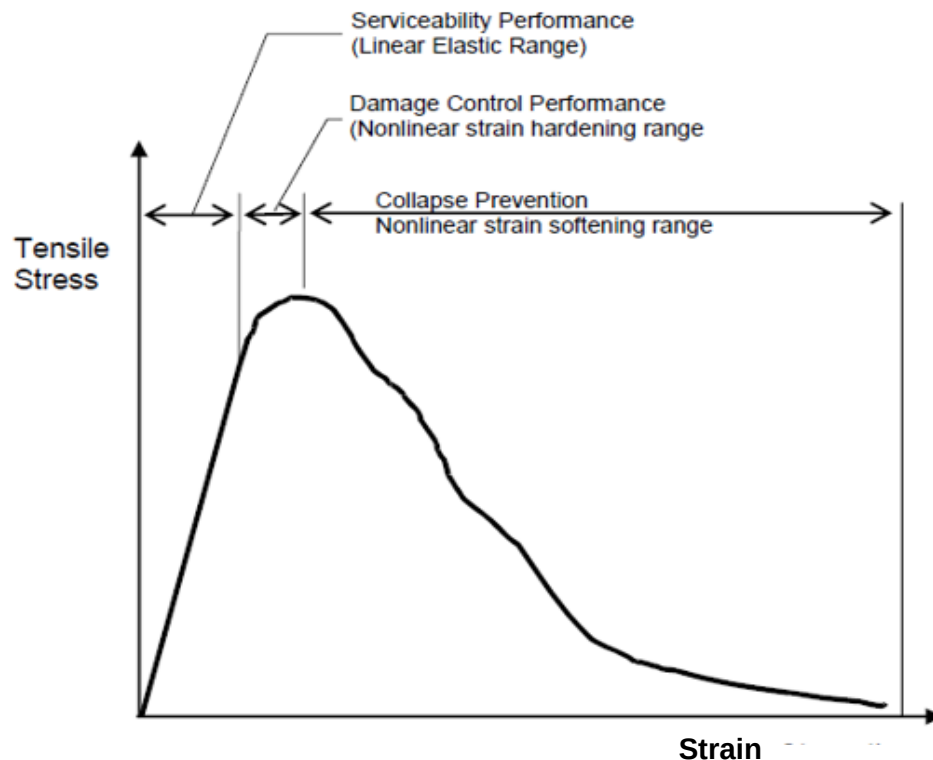


Figure 2-12 Stress-strain diagram and performance levels for plain concrete structures (USACE, 2007)

2.7.3 Dam Safety, Seismic Analysis and Assessment: ENTRO (2014) Guideline

(a) Dam Safety Regulatory Framework:

According to Eastern Nile Technical Regional Office (ENTRO,2014) Guideline, it is emphasized that a large number of the dams that are currently in place in the Eastern Nile countries are high risk, meaning that their failure might have disastrous effects. While Egypt is relatively in a better position in terms of having many laws addressing water management issues (irrigation, drainage, water quality management, electricity, energy, etc.), almost all Eastern Nile countries are lacking a well-articulated regulatory framework for dam safety. In the Ethiopian context, there is currently no explicit dam safety regulation that must be adhered to in the Ethiopian context for planning/designing, building, and operating dams. This is an essential issue that needs to be given utmost attention by the dam owners and responsible government agencies in the regions of the Eastern Nile Basin.

The ENTRO guideline (2014) further suggests that, in the absence of well structured and suitable national or regional dam safety regulatory framework, the ICOLD dam safety guideline would serve as more appropriate and suitable guidance to be adopted by the Eastern Nile countries.

It is also recommended in this guideline that periodic dam safety reviews be conducted using up-to-date information and techniques, which may differ from the accepted practices during initial construction or a previous dam safety review.

(b) ENTRO's (2014) Recommendations for Seismic Analysis of Dams

It is required that, for any dam to be designed and constructed in an earthquake-prone region, earthquake ground motions be appropriately considered using the most recent standards and techniques as basis for any analysis of the responses to seismic activities. For design of large dams and reservoirs, located in seismically active regions, or even in areas with little to no past seismic activity, site-specific seismic hazard studies are required to determine the probable seismic hazard levels necessary to assess the seismic potential and safety of the dam and its appurtenant structures.

According to this guideline, the seismic analysis should be based on the MCE ground motion, which should be determined using deterministic seismic hazard assessment (DSHA) method considering regional and local potential sources for seismic events. Additionally, for dams exceeding 100 meters high, or with storage capacity more than 1×10^9 cubic meters of water, the possibility of reservoir triggered-seismicity (RTS) needs to be considered.

2.7.4 Performane Evaluation and Acceptance Criteria: USACE (2007) Guideline

2.7.4.1 Performance Evaluation Procedures

The tensile behavior and cracking of large plain concrete structures often control their safety and serviceability. Massive concrete constructions actually have a very complex response to ground motions during earthquakes. The performance of a structure is significantly influenced by seismic strain rates and loading histories. Concrete is particularly sensitive to strain rate in terms of its ultimate tensile strength. The linear-elastic finite element method (FEM) of analysis is typically used to evaluate a concrete gravity dam. To guarantee that there won't be any or very little tensile cracking, the seismic evaluation method for gravity dams subjected to OBE ground vibrations is carried out within the linear-elastic range.

Gravity dams may respond inelastically to MDE ground motion demands if their performance is within the strain hardening range (also known as the damage control range in Figure 2-12). It is expected that cracking will start and spread at stresses greater than the ultimate tensile stress capacity. The time-history analysis results can be used to evaluate the damage potential of stress excursions that exceed the concrete tensile capacity as well as the potential implications of cracking on the dam's functionality.

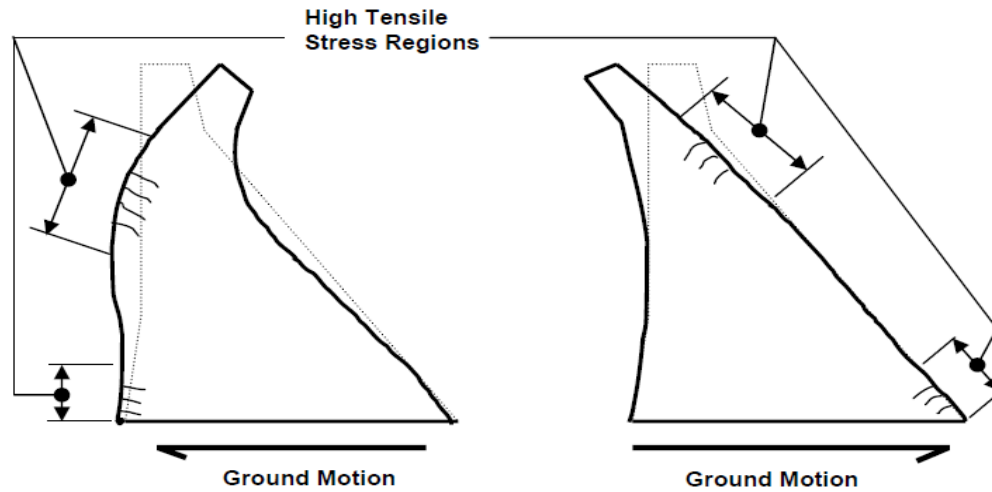
In general, the combined effects of static and seismic loads should be analyzed for the gravity dam model and the findings of the time-history analyses should include maximum stress envelopes, time histories of stresses, time histories of displacements, and time histories of reaction forces at the dam-foundation contact. To determine the severity and scope of overstressed regions, the maximum stress envelopes are used. The cumulative duration of stress excursions is calculated using the stress time histories and compared to the acceptability limits.

Demand-to-capacity ratio (DCR), along with the spatial extent of overstressed regions and cumulative duration of excursions above the tensile strength of the concrete, are used as performance indicators to begin the evaluation of seismic performance of concrete dams. DCR is used to evaluate the outcomes of response-spectrum analysis for concrete dams and is defined as the ratio of stress demands to static tensile strength of the concrete. Assessment of the outcomes of a linear time-history analysis is done using performance threshold curves (USACE 2007, EM 1110-2-6053).

Figure 2-13 along with its brief explanation in Table 2-17 below describe the behavior of a gravity dam in response to internal stresses and displacement during earthquake ground motion cycles in upstream-downstream and downstream-upstream directions.

In general, the tensile stress-strain results from linear elastic FEM are used to determine if the structure meets established project performance requirements.

- Performance under OBE loading conditions should be in the linear-elastic range (Serviceability Performance) as illustrated on the tensile stress-strain diagram of Figure 2-12.
- Performance under MDE /SEE loading conditions should be within the non-linear strain hardening range (Damage Control). The strain softening range provides reserve capacity against collapse and represents the concrete capacity to absorb additional energy demands from earthquake ground motions.



(a) Ground motion in downstream-upstream direction (b) Ground motion in upstream-downstream direction

Figure 2-13 Response of concrete gravity dam to horizontal earthquake ground motions (USACE, 2007)

Table 2-17 Response of concrete gravity dams to earthquake ground motions

Direction of horizontal earthquake motion	High tensile / potential cracking region	Reservoir loading condition	Reference
Downstream-upstream direction	Heel and upper part of upstream face	Maximum (operating) water level (MWL)	USACE (2007)
Upstream-downstream direction	Toe and slope discontinuities on downstream side	Minimum operating level or empty reservoir condition	

(Source: Reproduced from USACE, 2007)

2.7.4.2 Acceptance Criteria for Linear-Elastic Time-History Analysis

According to USACE (2003), it is commonly acknowledged that tensile stresses shouldn't be greater than the concrete tensile strength. However, on the basis of sound engineering judgment and practical considerations, up to five stress excursions above the tensile strength of the concrete have been accepted as appropriate. Nevertheless, the USACE manual (2003) proposed an updated acceptance criteria for the linear-elastic time-history analysis of concrete dams based on the use of performance threshold curves which take into account load combination cases, magnitudes of demand-capacity ratios (DCR's), the cumulative inelastic duration of stress excursions beyond the tensile strength of the

concrete, and the spatial extent of overstressed regions (Ghanaat, 2004; USACE 2003, EM 1110-2-6051).

Demand-Capacity Ratios (DCRs): The DCRs are calculated as the ratio of stress demands to static tensile strength of the concrete. Principal stress demands are used in the DCR computation for gravity dams. According to Raphael's (1984) formula, the static tensile strength of plain concrete, f_t' , is obtained from the uni-axial splitting stress tests or from the tensile strength test results:

$$f_t' = 0.32 f_c'^{2/3} \quad (2.38)$$

where f_c' is the compressive strength of the concrete (in MPa)

As shown in Figure 2-14, the highest permissible DCR for a linear analysis of a dam is 2, which corresponds to a stress demand twice the concrete's tensile strength. When evaluating the outcomes of a linear dynamic analysis, the stress demand associated with a DCR of 2 corresponds to the so-called "apparent" dynamic tensile strength of the concrete, which is $0.64 f_c'^{2/3}$, as defined by Raphael (1984).

Cumulative Inelastic Duration: The cumulative inelastic duration refers to the total duration of stress excursions above the tensile strength of the concrete (a DCR of 1 or larger). This criteria is a better indicator of the level of damage than simply counting the number of stress cycles.

Performance curves that are bounded by a demand-capacity ratios, as illustrated in Figure 2-14 below, present the acceptable damage level based on linear-elastic time history analysis for concrete gravity dams (Ghanaat, 2014; USACE 2003, EM 1110-2-6051). The damage rating and performance criteria include the following:

- (a) **Minor or no damage case:** When the estimated stress DCRs are less than or equal to 1.0, the dam response to the MDE event is regarded as being within the linear-elastic range of behavior with little to no potential of damage.
- (b) **Acceptable degree of damage:** In the event that the estimated DCRs are greater than 1.0, it is assumed that the dam will exhibit nonlinear behaviors in the form of contraction joints that open and close and cracking in the horizontal joints (lift lines) and concrete. The level of nonlinear response or cracking is deemed acceptable when DCRs are less than 2.0, and limited to 15% of the dam cross-sectional surface area, and the cumulative duration of stress excursions for all DCRs between 1 and 2 falls below the performance threshold curves shown in Figure 2-14. A nonlinear time-history analysis would be necessary to estimate the damage more precisely when these performance parameters are not satisfied, or are only partially met with the nonlinear response increasing the seismic demand.
- (c) **Serious Damage:** Damage is deemed serious if $DCR > 2$ or cumulative overstress duration for all DCRs in the range of 1 to 2 exceeds the performance curves shown in Figure 2-14. It might be necessary in such conditions to perform a nonlinear time-history analysis.

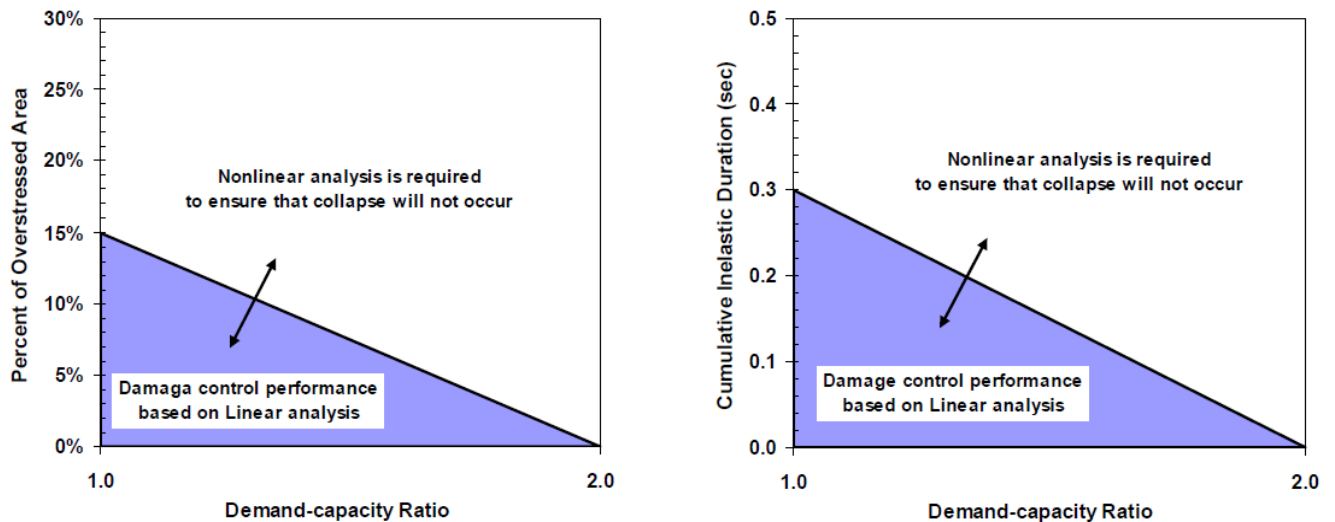


Figure 2-14 Performance /damage acceptance curves for concrete gravity dams (USACE, 2007)

According to Raphael (1984), for the linear-elastic time-history study, the ratio of the apparent dynamic tensile strength to static tensile strength, along with other parameters, will be utilized to evaluate the performance of existing concrete gravity dams. It is possible to apply stresses up to twice the static or at the level of the apparent dynamic tensile strength of the concrete (Figure 2-15). The "apparent" dynamic tensile strength of concrete (i.e., $f_t' = 0.64f_c'^{2/3}$), which is utilized for evaluating the outcomes of linear dynamic analysis, corresponds to the stress demand associated with a demand-to-capacity ratio of 2.

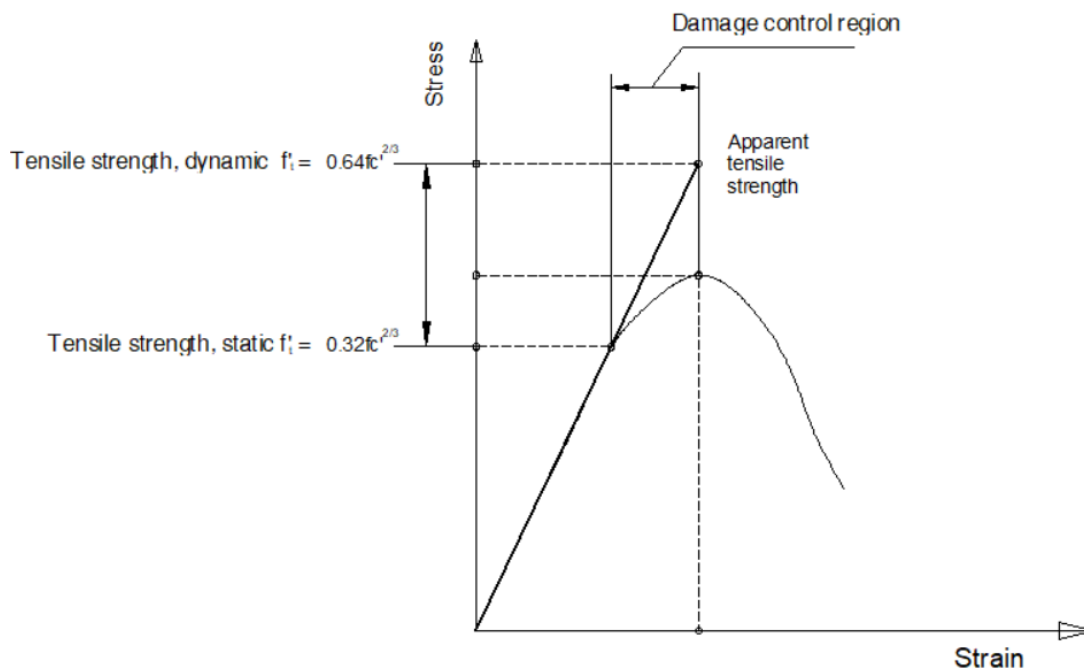


Figure 2-15 Apparent tensile strength of concrete for seismic response evaluation (USACE, 2003)

The effects of static loads as well as the vertical and horizontal components of earthquake ground motion should be considered while evaluating the load combination scenarios for two-dimensional models of gravity dams. The static loads and seismic components should be combined for each earthquake record in line with section 2.4.3.5, Table 2-15.

2.8 Methods of Earthquake Analysis

According to the USACE 2007 Engineer Manual EM 1110-2-6053, the evaluation of concrete gravity dams for earthquake ground motions should be carried out in stages, starting from the simplified methods such as the Sesimcic Coefficient and Equivalent Lateral Force methods, and advancing to a more complex methods of Linear Elastic Response-Spectrum and Time-History analysis, and finally, if necessary, nonlinear analysis.

The two most widely used earthquake analysis techniques are:

- 1) Static analysis method;
- 2) Dynamic analysis approaches

In this study, however, only dynamic analysis methods are briefly covered in below sections.

2.8.1 Dynamic Analysis Methods

In dynamic analysis methods, the structural response is determined based on the dynamic properties of the structure and the design earthquake ground motions defined for the specific dam site. The two dynamic analysis methods typically utilized are:

- Response spectrum analysis
- Time-history analysis:
 - Modal superposition
 - Direct integration.

2.8.1.1 Response Spectrum Analysis

According to this method, the design response spectrum and the dynamic properties of the structure are used to estimate the peak responses in each mode of vibration of linear elastic structures to earthquake ground motions represented by response spectra. For each analysis, all modes that significantly affect the structure's overall response should be considered. The overall mass participation of the modes utilized in the study must be at least within 90% of the total mass of the structure in order for the number of modes to be considered adequate. A computer program/software should be used to calculate the durations of vibration and mode shapes for all contributing modes. The maximum modal response is then estimated by combining these modal responses. The overall response can only be as large as the sum of the absolute values of each mode, whereas employing the square root of the sum of the squares results in a more likely total response. To get the total stress, dynamic stresses are added to the static stress.

For a variety of reasons, this method of analysis is recommended over the equivalent lateral force technique of analysis, which is constrained to a single mode. However, the response-spectrum modal analysis method has limitations in that, in most cases, it is sufficient for the dynamic response of the structure that are entirely within the linear elastic range; however, it does not account for varying stresses with time. Instead, time history dynamic analysis is typically advised for the final design and assessment of major structures like large storage gravity dams, and it provides reliable information for the anticipated inelastic response behaviors that may occur and justification for employing a nonlinear analysis (FEMA, 2005).

2.8.1.2 Time History Analysis

(a) Time History Mode Superposition

This method has similarity with the the linear-elastic-response spectrum / modal analysis approach discussed in section 2.8.1.1 above, with the exception that, in this method, earthquake demands are expressed as acceleration-time histories rather than response spectra, and the analysis findings are expressed in terms of displacement and stress histories. The response histories are used to extract the peak values of different response quantities. Unlike the response spectrum-modal analysis process, time history-modal analysis offers crucial time-dependent information that is missing in the former method. The time history-modal analysis is restricted to a linear elastic response, just like the response spectrum-modal analysis approach. The direct integration process is used in the time-history method to compute a structure's nonlinear response.

(b) Direct Integration Method

This is also a time history analysis approach that applies direct integration of the equations of motion, which can be used for both linear and nonlinear analyse. This advantage makes it the most effective technique for assessing the dynamic responses of structures to ground motions during earthquakes. It is a method for step-wise numerical integration of data that yields stresses (or forces) and displacements for a successive short-time increments from the start of loading to any target time of interest. For ease of computation, the time steps are typically taken to be equal in duration. At the start and finish of each time increment, the condition of dynamic equilibrium is solved; on the basis of an assumed response characteristics, the system's motion throughout each time interval is evaluated.

Some of the common direct integration methods widely used in dynamic analysis are discussed in section 2.12.2.1.

2.9 Analytical Modeling of Structural Systems

When developing the failure modes and the analytical models, it is important to take into account the dynamic interactions between the different structural elements. The analysis should pay particular attention to the interactions between the structure, its base, and any impounded, surrounding, or

contained water. However, the structural model chosen should be compatible with the level of detailing utilized to characterize the earthquake ground motion, and it is usually advised to start with the simplest model that can provide the necessary data and level of certainty for evaluation (FEMA, 2005).

2.9.1 Types of Structural Models

When developing structural analysis model, the structure geometry, stiffness, damping, and mass distributions must all be taken into account because they have an impact on the structural dynamic characteristics. To create a simple, accurate and representative model, engineering judgment and structural dynamics expertise are necessary. A concrete gravity dam may be represented by a planar or 2-D model with deformations limited in a plane or by a more comprehensive 3-D model to account for three-dimensional behaviors, depending on its nature and degree of complexity (FEMA, 2005).

(a) Two-Dimensional (2-D) Models

The analysis of a gravity dam has traditionally been done using a very basic mathematical model of the structure, where the resistance to external forces is only evaluated for a unit slice of the dam taken in an upstream-downstream orientation. The earthquake forces were simply considered as static forces being expressed as the product of a seismic coefficient and weight of the structure. The impacts of upstream-downstream horizontal ground motion were considered. However, it is currently common practice to use finite element modeling in the analysis of both the static and dynamic response of concrete gravity dams in order to understand the resistance mechanism properly.

Planar or very long structures, such as long gravity dams, which are composed of a monolith dam, flexible foundation rock and impounded reservoir, are usually modeled using the 2D finite-element idealization. Depending on whether the stress or strain can be disregarded in the out-of-plane direction finite elements, the highest monolith or dam cross section is often chosen and modeled using plane stress or plane strain elements. Two components of ground motion applied in the vertical and horizontal directions are typically investigated in a 2-D model (USACE 2003, EM 1110-2-6051).

(b) Three-Dimensional (3-D) Models

Arch dams, curving gravity dams, and structures erected in small canyons are examples of structures with complex or irregular geometry or nonuniform loads that can be studied using 3-D finite element idealization (USACE 2007, EM 1110-2-6053).

2.9.2 Approaches for Analytical Modelling of Concrete Gravity Dams

The USACE (2003), Engineer Manual EM-1110-2-6051, recommends two methods for the analytical modeling for time-history analysis of concrete gravity dams based on the effects of fluid-structure interaction and foundation-structure interactions:

- 1) The substructure method
- 2) The standard finite element

2.9.2.1 The Substructure Method

(a) Modeling Approach

This method is a frequency domain analysis approach in which the foundation rock for 2-D analysis is idealized as a viscoelastic half plane. This method is applicable in situations where the foundation rock is composed of essentially identical (homogeneous) rocks that reach to large depths or cover the entire unbounded foundation region. In this way, the entire system is decomposed into three substructure units: the structure, the water, and the foundation rock (Figure 2-16 in page 81).

(b) Dam

The dam structure is normally represented as a beam or 2-D finite element plane-strain system.

(c) Reservoir

The water domain may be idealized as a continuum or as a combination of a finite element and continuum system.

(d) Impedance Function:

The impedance function, which is defined with respect to the nodal points at the interface between the foundation rock and the dam, describes the stiffness and damping properties of the foundation-structure interaction in a viscoelastic halfplane (2-D) model. According to Dasgupta and Chopra (1979), the impedance matrix for the viscoelastic half plane is derived from an independent continuum solution of the foundation rock region.

(e) Interacting Forces at Dam-Foundation Interface:

The dynamic interaction between the structure and the water is expressed as frequency-dependent hydrodynamic forces at the structure-water interface, assuming that the reservoir water can be modeled as a fluid domain with constant depth and infinite length in the upstream direction. The free-field acceleration time histories applied at the dam foundation and at the reservoir-foundation interface zones are included in the seismic input as a horizontal and vertical component.

The interaction between the foundation and the structure adds flexibility at the structure base and provides additional damping mechanisms through material damping and radiation. The flexible foundation tends to prolong the period of vibration, and the material and radiation damping in the foundation region has the effect of lowering the structural response. Interacting forces at the structure-foundation interface (i.e. base nodes) represent the dynamic interaction between the structure and the foundation. These interdependent forces, which change in frequency, are related to the displacements produced by the dynamic stiffness (impedance) matrix for the foundation region.

(f) Foundation:

The undelying foundation region may be idealized as a continuum (Dasgupta and Chopra 1979) or as a finite element system. When similar materials stretch to significant depths, the continuum idealization enables realistic simulation of the structure-foundation relationship.

This substructure method is not covered in detail in this study and presented for information purpose only.

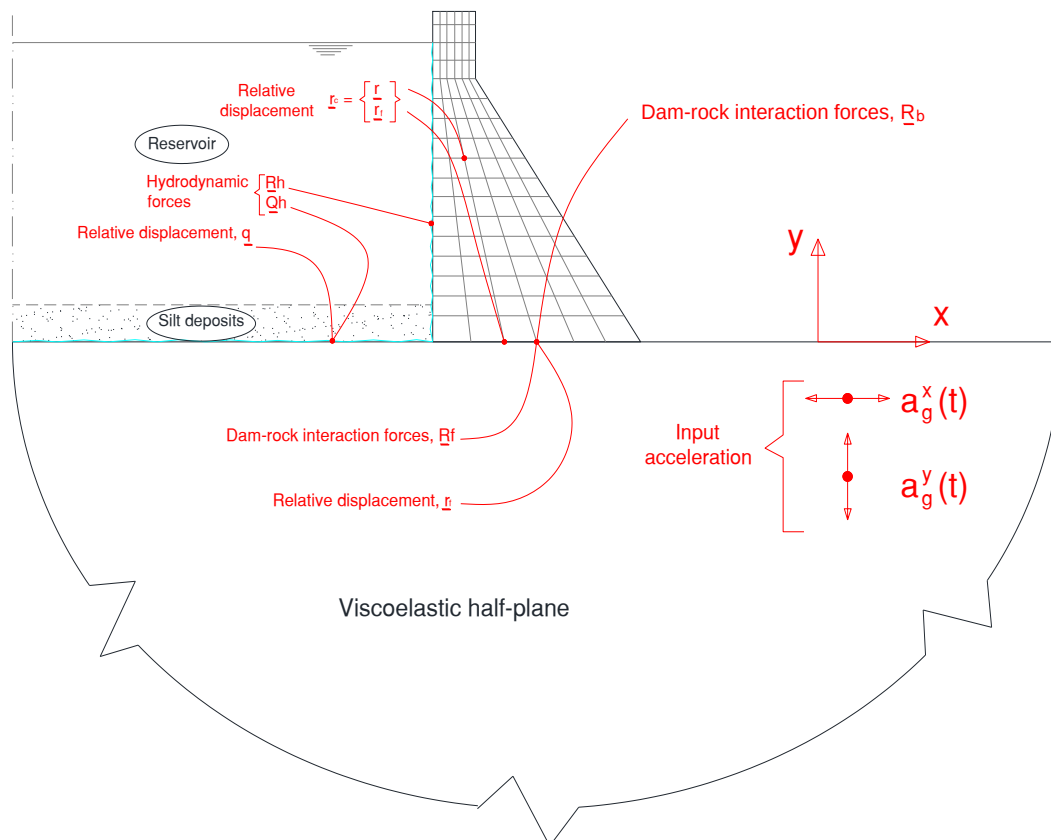


Figure 2-16 Substructure model of the dam-water-foundation system (USACE, 2003)

2.9.2.2 Standard Finite Element Method

A time domain analysis strategy is used in this procedure. In situations where foundation rock properties vary with depth and along the foundation axis of the dam,. a finite element idealization that allows for the changing of rock characteristics with depth can be taken into consideration in instances when it is necessary to assign various rock properties to different layers since the viscoelastic half plane model is inapplicable.

The entire structure-water-foundation system is modeled and analyzed as a single composite structural system as illustrated in Figure 2-17 below, The structure is modeled as an assembly of beams or finite elements, much like the substructure technique. Typically, simple models that only describe the water-structure and structure-foundation interactions are used to represent them. The hydrodynamic pressures are represented by lumped mass of water attached to the dam, incompressible water is assumed. Added mass of water is used to represent the hydrodynamic forces and water compressibility is ignored. According to A.K. Chopra (2017), the technique disregards radiation dampening into reservoir bottom sediments, impounded water, and foundation rock.

USACE 2003, EM 1110-2-6051 suggests that a massless finite-element foundation model of the foundation rock above a rigid horizontal boundary is the most simple approach to represent the dynamic interaction effects of dam-foundation interfaces The following assumption of massless foundation model helps in simplifying the application of the design seismic acceleration inputs and avoiding the use of large foundation models.

- The foundation rock region is represented by a finite element model that ignores the inertia and damping effects of the foundation and only accounts for its flexibility.
- The modulus of deformation (E_d), which for a case of linear-elastic analysis is equivalent to Young's modulus of elasticity (E_f), describes the flexibility of the jointed rockmass foundation.
- The massless foundation mesh should be extended in the upward, downward, and downstream directions by a length at least equal to the height of the dam.
- The nodal points at the bottom of the foundation mesh are fixed in both the vertical and horizontal axes. However, for the horizontal and vertical excitation of the dam, the side nodes are attached to horizontal roller supports and vertical roller supports respectively
- The earthquake ground motions recorded at the ground surface are directly used as the seismic input and are applied at the rigid base (fixed boundaries) of the massless foundation model.

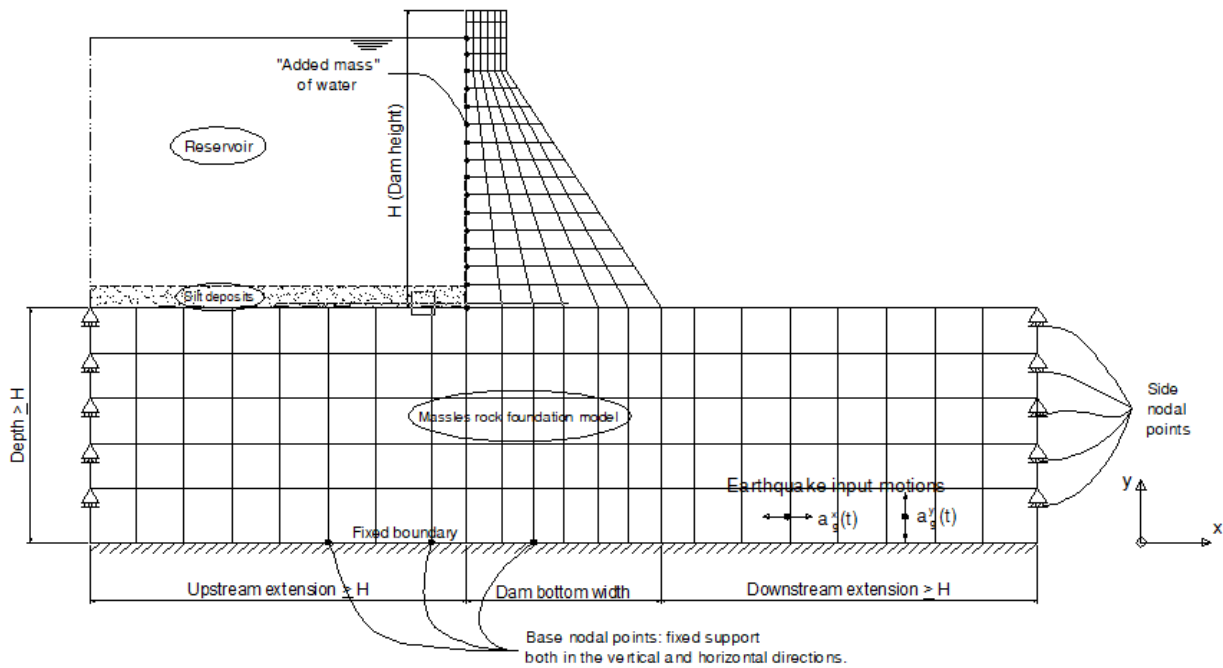


Figure 2-17 Standard finite element model of the dam-water-foundation system (USACE, 2003)

According to Chopra, A. (2014), despite its simplicity, with the assumption of massless foundation with flexible foundation, and material damping and radiation damping ignored, the degree of overestimation of crest displacement and maximum principal stresses increases as E_f/E_s (the ratio of elastic moduli of foundation rock and dam concrete) decreases, i.e., the foundation becomes softer.

2.10 Numerical Modeling of Dam-Reservoir-Foundation System

2.10.1 Dam-Reservoir Interactions: Incompressible water case

2.10.1.1 The Generalized Westergaard's Added Hydrodynamic Mass

As discussed in section 2.5.9.3, Kuo (1982) developed the Generalized Westergaard Added Mass idea that is thought to be appropriate for general or arbitrary geometry of upstream dam face (sloped or curved) and takes account of the flexibility of the dam by assuming that the hydrodynamic pressure at any node 'i' on the upstream face is proportional to the total acceleration acting normal to the dam face at that particular node. The method is used for the parabolic hydrodynamic pressure distribution over the upstream face of the dam based on the concept of added mass (similar to the original Westergaard model), incompressible water added as lumped mass on the upstream side, and foundation stiffness considered to obtain the analytical solution for the hydrodynamic pressure distribution. Through interaction forces and suitable continuity conditions, the fluid domain and the dam are coupled at the dam upstream face.

Despite the limitations in the application of this method in the preliminary analysis and design of concrete dams, this simplified coupled analysis methodology has regularly been employed in engineering practice (USACE 2003, EM 1110-2-6051). Malm, R. (2016) also suggests that, the

Generalized Westergaard added mass model is commonly regarded sufficient to represent the hydrodynamic pressures on the dam face in dynamic response analysis.

In this method, the effects of surface waves are frequently ignored, and hence zero pressure is assumed at the free water surface. The pressure direction varies along the different points on the a curved or sloped 2-D dam face. Using the finite element formulation, for a given tributary area, A_i , on the upstream face, the added mass, m_{ai} , at any node 'i' on the upstream face of a 2-D dam structure can be expressed in matrix form in Cartesian coordinates (Kuo, 1982) as follows:

$$m_{ai} = \alpha_i \cdot A_i \cdot \lambda_i^T \cdot \lambda_i = \alpha_i \cdot A_i \begin{bmatrix} \lambda_x^2 & \lambda_y \lambda_x \\ \lambda_y \lambda_x & \lambda_y^2 \end{bmatrix} \quad (2.39)$$

where

m_{ai} = added mass matrix at any node 'i' on the upstream face of dam

A_i = tributary area associated with node 'i'

λ_i = vector of normal direction cosines to the surface at node 'i'

$$\lambda_i = \begin{bmatrix} \lambda_x \\ \lambda_y \end{bmatrix} \quad (2.40)$$

α_i = Westergaard pressure coefficient, given by:

$$\alpha_i = \frac{7}{8} \cdot \rho_w \cdot \sqrt{H_i(H_i - z_i)} \quad (2.41)$$

ρ_w = mass density of water (981kg/m³)

H_i = depth of water at the vertical section that includes node 'i'

Z_i = height of node 'i' above the base of the dam

Based on the above assumptions, the equations of motion for the complete 2-D dam-foundation-reservoir system subjected to earthquake ground motion can be written as:

$$(m_s + m_a)\ddot{u} + c\dot{u} + ku = -(m_s + m_a)1_s^x \ddot{u}_g^x(t) - (m_s + m_a)1_s^y \ddot{u}_g^y(t) \quad (2.42)$$

with physical meaning interpreted as,

$$\boxed{\text{Inertia forces} + \text{Damping forces} + \text{Elastic forces} = \text{Effective earthquake loads}}$$

where

m_s = mass matrix of the structure

m_a = added hydrodynamic mass matrix having nonzero terms only at the structure-water nodal points

$\dot{u}$ = velocity vector

$\ddot{u}$ = acceleration vector

c = overall damping matrix for the entire system

k = combined stiffness matrix for structure and foundation region

u = vector of nodal point displacements for the complete system relative to the rigid base

displacement

1_s^x = vector of 1's corresponding to x-DOFs

1_s^y = vector of 1's corresponding to y-DOFs

(Note: 1_s^x and 1_s^y represent the influence coefficient vectors corresponding to x- and y-directions, having 1 for elements corresponding to degrees of freedom in the direction of the applied ground motion and zero for the other degrees of freedom).

$\ddot{u}_g^x(t)$ = ground acceleration input in x-direction

$\ddot{u}_g^y(t)$ = ground acceleration input in y-direction

Due to the hydrodynamic pressures acting normally at the structure-water interface, the added hydrodynamic mass typically includes nonzero terms for the x- and y-DOFs. Simple geometry at the structure-water interface may result in zero additional hydrodynamic mass terms for some DOFs. For a gravity dam with a vertical upstream face, for instance, only the added hydrodynamic mass terms corresponding to the x-DOFs (horizontal direction) are nonzero. Because the inertia and damping effects of the foundation rock are disregarded, the foundation rock contributes only to the stiffness matrix.

According to Malm, R.(2016) the Generalized Westergaard added mass approach is commonly regarded sufficient to represent the hydrodynamic forces in most cases of dynamic response analysis.

2.10.1.2 Finite Element Added Hydrodynamic Mass Model

According to USACE 2003, EM 1110-2-6051, the Westergaard approach is computationally efficient, however, for more in-depth analysis of concrete gravity dams with complicated or arbitrary geometry, neither the original Westergaard method (1933) nor the Generalized Westergaard formula (Kuo, 1982) are often suitable. A more realistic idealization of the structure-water interface and the reservoir bottom is possible for such structures when the incompressible fluid domain is idealized using finite elements.

(a) Equations of Motion

It is obvious that the reservoir medium will become excited and begin vibrating as seismic waves pass through the reservoir water (Chowdhury, I. & Dasgupta, 2019). The Laplace Equation governs the small-amplitude motion of water caused by the hydrodynamic pressure, p , for an incompressible and inviscid fluid in 2-D system (Ghanaat, Y., as cited in USACE 1993), as given by:

$$\nabla^2 p = 0 \quad (2.43)$$

$$\frac{\partial^2 p}{\partial x^2} + \frac{\partial^2 p}{\partial y^2} = 0 \quad (2.44)$$

where

∇^2 = Laplacian operator in 2-dimensions

$p(x,y,t)$ = hydrodynamic pressure, in excess of the static pressure in the fluid domain generated by acceleration of the structure-water interface and acceleration of the reservoir bottom.

(i) Reservoir Boundary Conditions

Hydrodynamic pressure is produced in the dam-foundation-reservoir system when the boundaries in contact with the reservoir are accelerated. As shown in Figure 2-18 the reservoir interacts with the dam on the face, S1, and with the rock foundation on the bottom face, S2. To estimate the hydrodynamic pressures acting on the structure-water interface, Eqs.(2.44) needs to be solved under the assumption that the effects of surface waves and fluid viscosity are ignored.

(ii) Reservoir free surface:

It is assumed that the effects of surface waves, which are known to be minimal, are neglected and also the pressure along the free surface is equal to zero.

$$p(x,y,t) = 0 \quad (2.45)$$

(iii) Dam-reservoir interface:

At the upstream face (structure-water contact surface), the normal acceleration of the water is the same as the total normal acceleration of the dam, and hydrodynamic pressures, p , are related to the total accelerations, $\ddot{u}_{nd}$ with the boundary condition given by:

$$\frac{\partial p(x,y,t)}{\partial n} = -\rho_w \ddot{u}_{nd}(x, y, t) \quad (2.46)$$

where

ρ_w : the mass density of water

$\ddot{u}_{nd}$: the component of total acceleration normal to upstream face of the dam

p : the hydrodynamic pressure

ϕ : the velocity potential

n : represents the direction normal to the boundary (positive when pointing towards the dam face)

USBR (2006) recommends the use of fluid-like elements for reservoir-dam interaction modeling, in which case a two-dimensional mesh of fluid elements can be developed to represent the reservoir in a typical analysis of the dam-reservoir interactions, and the interface between the reservoir and the dam can either be a tied contact or a sliding contact surface.

(iv) Reservoir bottom boundary:

The presence of silt deposits (or sediments) at the reservoir bottom has been found to significantly affect the seismic distress and response of concrete gravity dams because the soil layers do not behave as totally reflective boundaries like rigid rock and there is dynamic interaction between the reservoir and the underlying soil (Papazareopoulos et al., 2011). Hence, a similar boundary condition may be considered.

$$\frac{\partial p(x,y,t)}{\partial n} = -\rho_w \ddot{u}_{nb} \quad (2.47)$$

where $\ddot{u}_{nb}$ is the normal component of ground acceleration at the reservoir boundary.

Ghanaat (1993) suggests that the pressure on the dam as a result of motion of the reservoir boundary, S2, can be ignored as the reservoir boundary does not move.

USBR (2006) recommends using fluid-like features to simulate the reservoir-foundation interaction in which case the reservoir can be represented as a two-dimensional mesh of fluid elements that is connected to the foundation by a tied contact surface.

(v) Far-field reservoir: truncation boundary conditions

Along with the above boundary conditions, a reservoir of infinite extent cannot be used for the finite element discretization of the reservoir water. The reservoir model must be truncated at a suitable distance from the dam upstream face so as to create bounded reservoir in the far-field of the unbounded fluid domain. The following boundary condition is applied to this boundary:

$$\frac{\partial p(x,y,t)}{\partial n} = -\rho_w \ddot{u}_{nu} \quad (2.48)$$

where $\ddot{u}_{nu}$ is the acceleration normal to the upstream (truncated) boundary

Ghanaat (1993) suggests that, for an upstream extent of reservoir larger than the three times the dam height, the effect of the hydrodynamic pressure on the face of the dam is so small that it can be neglected in practical applications.

$$\frac{\partial p(x,y,t)}{\partial n} = 0 \text{ or } p(x,y,t) = 0 \quad (2.49)$$

A typical two-dimensional physical model of the dam-reservoir system of a concrete gravity dam with sloping upstream face is shown in Figure 2-18 below.

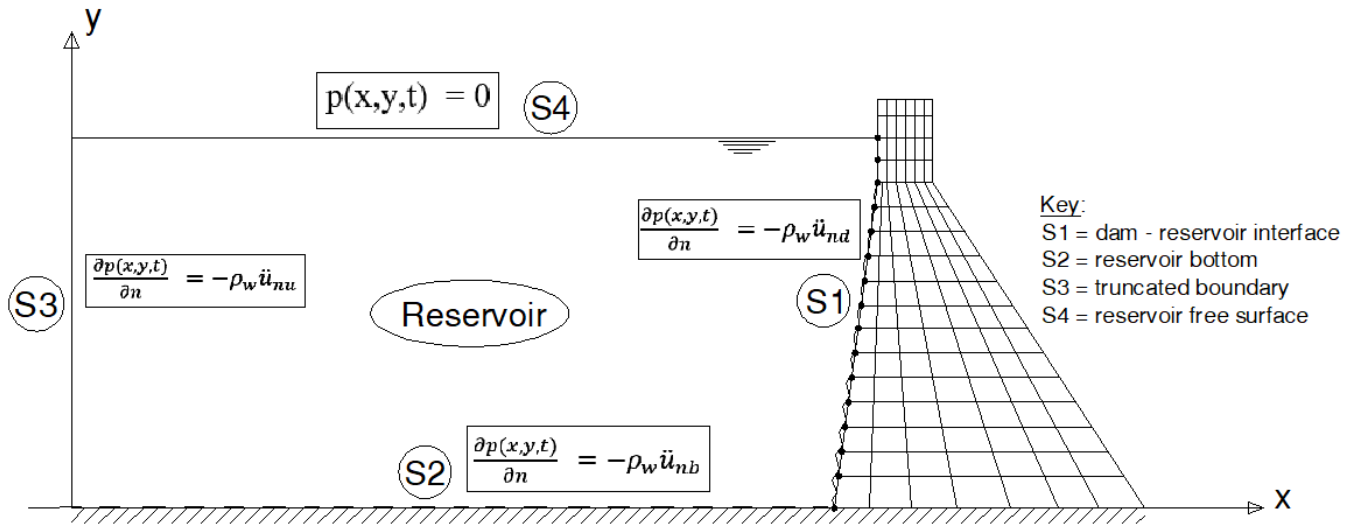


Figure 2-18 Schematic of dam-reservoir finite element domain with reservoir boundaries

2.10.2 Dam-Foundation Interactions

2.10.2.1 Massless Finite Element Foundation Model

As advised by USACE 2003, EM 1110-2-6051, the most simple method to represent the effects of dam-foundation interactions is to include the foundation rock region above a rigid (fixed) horizontal barrier in the finite element idealization. Then, using the standard procedures, the response to the earthquake excitation applied at the rigid base is calculated. However, this method can result in very large foundation models with similar subsurface materials extending to great depths and there is no clearly defined margin for the "rigid" boundary limit to set the fixed base. These challenges can be resolved by using a simplified massless foundation model, in which the foundation rock inertia and damping effects are ignored and only its flexibility (stiffness) is considered in the analysis.

In this approach, the inertia force is only applied to portions of the model with defined masses, the frequencies and response of the dam only are computed, and they are independent of the size of the foundation model. However, crucial inertia effects and radiation damping are disregarded when a massless foundation is used. Ignoring radiation damping could cause the dam to experience unreasonably large stresses (USBR, 2006). A massless foundation model doesn't need to be particularly so large as long as it gives a good approximation of the flexibility of the foundation rock region supporting the dam. Most of the time, a foundation model that extends one dam height upstream, downstream, and downward will be sufficient.

Advantages of massless foundation model (USACE 2003, EM 1110-2-6051):

- This approach allows distinct rock properties to be assigned to different elements or strata, in contrast to the homogenous viscoelastic half plane model. The variation of rock characteristics with depth is considered.

- The massless foundation model also enables the direct application of the earthquake ground motions recorded on the ground surface at the base (fixed) boundaries of the foundation model. This is due to the fact that in the absence of wave propagation, the motions of the fixed boundaries are directly transferred, without any changes, to the dam base.

2.10.2.2 Dam Structure – Foundation Interface

Chavez, J. & Fenves (1993) suggest that the dam-foundation interface may be modelled on the basis of the assumption that the interface is a straight surface having frictional resistance against sliding. The Mohr-Coulomb failure criteria is used to define cohesion and angle of internal friction at the interface.

2.10.3 Dam-Reservoir-Foundation System

Currently, due to the significant advancements in the computational power of the finite element method, two-dimensional finite element models are primarily used to analyze concrete dams. In these models, the entire dam-reservoir-foundation system model, subject to seismic loading, is discretized into an assembly of smaller plane-strain finite elements. An equilibrium equation is established for every element, which is then put together to create the entire model. The analysis results are obtained by establishing boundary conditions for the full domain and solving the system of non-linear equations.

The finite element formulation of the dam-reservoir-foundation system is done in such a way that the discretized dynamic equations of the fluid and structure, including the dam and its foundation, are taken into account at the same time in order to produce the coupled fluid-structure-foundation system. The following approaches are the most commonly used for finite element modeling of the complete dam-reservoir-foundation system (Chowdhury, I. & Dasgupta, P., 2018).

- (1) **Finite element model approach-1:** this approach involves the assumption of incompressible water added as lumped mass on upstream face, foundation rock fixed at base, and foundation stiffness considered but damping effects neglected (Figure 2-19).
- (2) **Finite element model approach-2:** this approach is a comprehensive dam-reservoir-foundation interaction analysis considering the dam, fluid, and foundation discretized into an assemblage of smaller plane-strain finite elements and their interactions at interfaces (dam-reservoir, dam-foundation, reservoir-foundation) defined by appropriate boundary conditions (Figure 2-20).

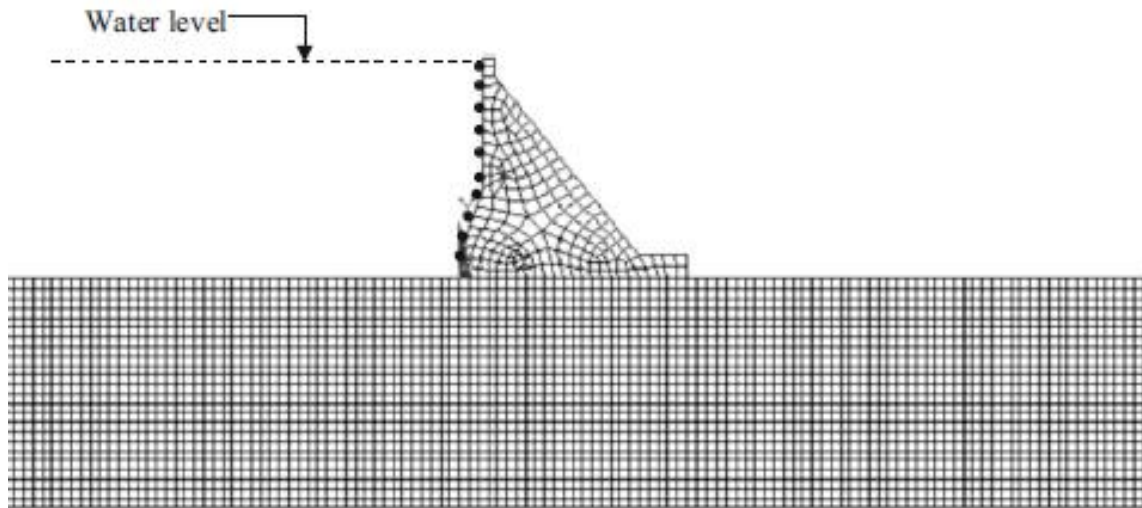


Figure 2-19 Finite element model of dam (fixed base) with full reservoir: incompressible water added as lumped mass on upstream side and foundation stiffness considered (Reproduced from Chowdhury, I. & Dasgupta, S.P., 2019).

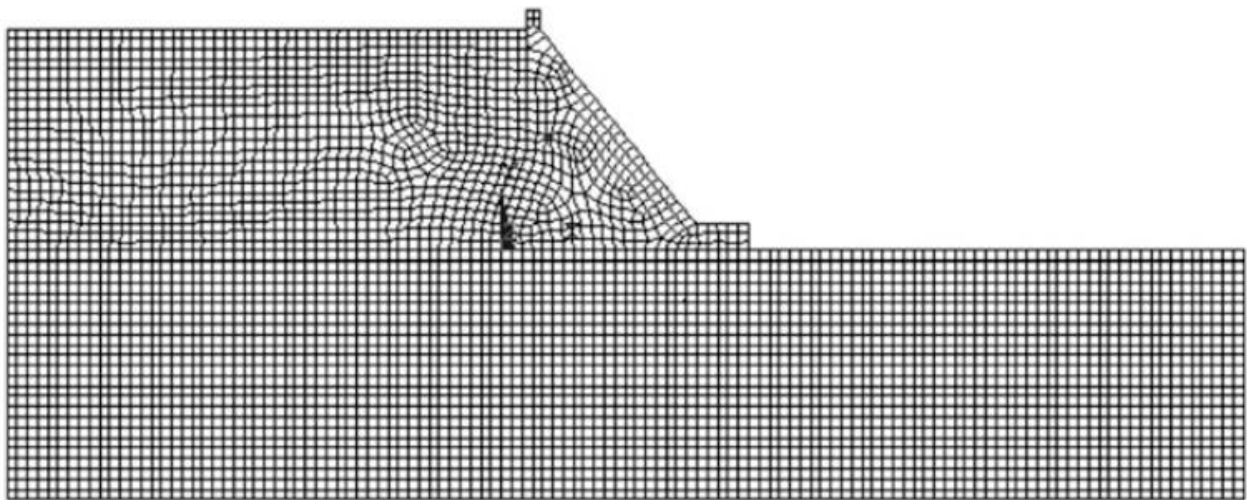


Figure 2-20 Finite element model of dam-fluid-foundation system for a comprehensive foundation-fluid-structure interaction analysis (Reproduced from Chowdhury, I. & Dasgupta, S.P., 2019).

Of the two approaches discussed above in (a) and (b), it is clear that the numerical computation method with finite element modeling approach using a discretized assemblage of the dam-reservoir-foundation system, illustrated in Figure 2-20, is most complete and frequently used in the dynamic analysis of concrete dams (Chowdhury, I. & Dasgupta, S.P., 2019)

2.10.4 Effective Stiffness

According to USACE (2007) Engineer Manual EM 1110-2-6053, it is likely that the elastic limit of concrete would be exceeded for concrete gravity dams when subjected to strong earthquake ground shaking, which might result in some cracking and possibly a decline in the overall stiffness of the structural system. The overall stiffness of such large structures will be hardly affected if cracking develops close to their surfaces. As a result, it is appropriate to employ stiffness parameters related to

the uncracked section properties in linear elastic analysis. However, nonlinear analyses should be used for evaluation in the event that the cracking is widespread and substantial.

2.10.5 Effective Damping

Damping is the process by which the amplitude of a structure's vibration gradually decreases due to energy loss in the vibrating system caused by a variety of mechanisms, including internal and external material friction, interaction of parts, plasticity, viscosity, hydrodynamic(fluid) interaction, radiation, and other factors. Damping is not something that can be measured directly; it is rather estimated by testing or specified by industry practices as it is usually present in most systems and needs to be specified during dynamic analysis. .

The dissipation of energy in foundation-structure system can be described by two sources:

- Radiation damping - the dissipation of energy in the boundaries, and
- Material damping - the loss of energy within the foundation material itself.

Dams are among the important structures that typically have a damping ratio of less than 10%. Most representations of damping in real structures are highly idealistic. Therefore, equivalent viscous damping is the name given to this idealization (Chopra, 2012). Also, Wieland (2012) argued that damping is the main factor determining a concrete dam's dynamic reactions and that damping ratios of less than 7% be employed for gravity dams in the absence of background information or reliable data in the evaluation of the SEE ground motion.

Calculating an effective viscous damping ratio to account for the damping contributions of both the dam and the foundation is necessary since the material and radiation damping of the foundation considerably contribute to the damping of the combined dam-foundation system (USACE 1995, EM 1110-2-12).

However, an effective damping of 5% of the critical provides an acceptable estimate of the dynamic response of concrete gravity dams at or near yield and cracking. However, damping may be as low as 2 to 3 percent for loads well below yielding and cracking and as larger than 5 percent if the structure exhibits energy dissipation through joint opening, tension cracking, and yielding. Unless the dynamic interaction effects between the structure and foundation, as well as between the structure and impounded water, are significant to justify the use of higher effective damping values between 5 to 10 percent, the damping value should generally be limited to 5-percent (USACE 2007, EM 1110-2-6053).

In a full transient dynamic analysis using direct time integration methods, Rayleigh damping, as defined by Eqs.2.42 as well as material dependent damping can be specified (Berrabah, T., 2012).

$$[C] = \alpha[M] + \beta[K] \quad (2.50)$$

where $[M]$ and $[K]$ are the mass and stiffness matrices respectively and α and β are constants of proportionality corresponding to material and stiffness damping coefficients.

2.11 Linear-Elastic Analysis of Two-Dimensional Problems

2.11.1 Finite Element Analysis Basics

In engineering, the finite element method (FEM), also known as finite element analysis (FEA), is a numerical (computational) method of analysis used to compute approximations of solutions to boundary value problems. The FEA is a method of approximating continuum problems in which the continuum is divided into a finite number of elements with behaviors specified by a finite number of parameters, and the solution of the complete system as an assembly of its elements follows precisely the same rules as those applicable to standard discrete problems (Hutton, D., 2003; Zienkiewicz et al., 2013).

The finite element analysis of the structural system is performed on the basis of the following principles or engineering philosophies (USACE, WES, 1993),

- (i) The entire structure is idealized as an assemblage of discretized finite elements that are connected to each other at discrete number of nodes having a finite number of unknowns.
- (ii) The primary unknown variables are represented by the displacements of the nodal points, which are located at the borders of each finite element.
- (iii) A set of presumptive functions that represent displacements within the element region in terms of the element nodal displacements are used to approximate the behaviour of each finite element. These displacement functions are frequently represented in a way that guarantees continuity across adjacent elements and, consequently, across the entire structure.
- (iv) Since displacements are known at all nodes of the element, the strains at any node can also be calculated in terms of nodal displacements. The state of stresses throughout the element and its boundaries are defined by these strains along with any initial strains.
- (v) The stiffness of each element can be computed by meeting the force-equilibrium condition between a system of equivalent nodal forces with the boundary stresses and any distributed load operating on the element using the relationships between strain, displacements, and stress-displacements.
- (vi) In the end, the individual elements are then combined in a way that satisfies the requirements of equilibrium condition and compatibility at the junctions of these elements to provide the equilibrium equations for the complete system. Numerical techniques are used to solve a system the equilibrium equations.

When evaluating the response behaviors of a concrete gravity dam during an earthquake, it is common to assume that the structure will respond linearly to an earthquake of weak to medium intensity. This means that the deformations are anticipated to be directly correlated with the magnitude of the

earthquake applied to the foundation. The assumption that the structure will respond linearly helps in simplifying the formulation of the mathematical modeling of the dam, the reservoir, and the rock foundation, as well as the process for analyzing the seismic responses.

The outcomes of the linear analysis reveal the overall properties of the dynamic response, and the magnitude of the computed displacements and strains shows whether the linearity assumptions are realistic or not. However, it is likely that during a strong earthquake ground motion, the calculated deformations will exceed the elastic strength characteristics of the concrete and result in damages to the dam, which can only be assessed using a more advanced nonlinear analysis.

The FEA of the responses of concrete dams to earthquake effects should take into account the dam-water-foundation interactions in the reservoir, the absorption of the pressures caused by seismic waves within the boundaries of the reservoir, and the dam-rock interactions in the foundation.

2.11.2 Boundary Conditions

In a boundary value problem, one or more dependent variables must satisfy a differential equation everywhere within a known domain of independent variables and satisfy specific conditions on the boundary of the domain.

The boundary conditions are the predetermined values of the field variables (or closely related variables, like derivatives) on the boundaries of the field where the structure interacts with the environment either through the application of an external force or through some constraint that is imposing a displacement (Hutton, D., 2003). In this approach, boundary conditions represent constraints that are defined before the analysis even begins and required to resolve a boundary value problem.

Every point on the boundary of a structure must have a known boundary condition, such as a known force or displacement, in order to obtain solution of solid mechanics problem. It is possible for the known force or displacement to be zero or to have some value. There are two types of boundary conditions:

- (1) **Essential (Dirichlet) boundary conditions:** These are displacement (essential) boundary conditions that specify the values that a solution needs to take along the boundary of the domain.

$$u = \bar{u} , \quad v = \bar{v} \quad (\text{on displacement boundary}) \quad (2.51)$$

where $\bar{u}$ and $\bar{v}$ are prescribed values for the displacement components.

- (2) **Natural (Neumann) boundary conditions:** These are force boundary conditions that specify the values of the derivative of the dependent variable applied normal to the boundary of the domain and given in terms of stresses by

$$t_x = \bar{t}_x, t_y = \bar{t}_y \text{ (on force boundary)} \quad (2.52)$$

where t_x and t_y are tractions (stresses on the boundary) with known values.

Both boundary condition types are vectors, and it is possible to specify some boundary components using displacement conditions and others using force conditions at any point along the boundary.

2.11.3 Initial Conditions

Initial conditions are conditions that are necessary to define the state of the structure at the beginning of the analysis. While a second time derivative seems to represent the acceleration, it is important to set the initial displacement and velocity (Zienkiewicz et al. (2013)).

$$\left. \begin{array}{l} \text{Initial displacement:} \quad \mathbf{u}(\mathbf{x}, 0) = \bar{\mathbf{d}}(\mathbf{x}) \\ \text{Initial velocity:} \quad \dot{\mathbf{u}}(\mathbf{x}, 0) = \bar{\mathbf{v}}(\mathbf{x}) \end{array} \right\} \quad (2.53)$$

2.11.4 Linear Elasticity Equations

In a 2-D linear elasticity theory, the materials constituting the structural body are considered to be linearly elastic, isotropic, and homogenous for the purposes of stress and deformation analysis. Displacements, strains, stresses, boundary conditions, initial conditions, and constitutive equations that correlate the stress and strain behaviour are used to describe the fundamental equations for the theory of elasticity.

In general, there are six independent components of stresses and strains at any point in a 3-D structure,

$$\left. \begin{array}{l} \text{Stress:} \quad \{\sigma\} = [\sigma_{xx}, \sigma_{yy}, \sigma_{zz}, \tau_{xy}, \tau_{yz}, \tau_{zx}]^T \\ \text{Strains:} \quad \{\varepsilon\}^T = [\varepsilon_{xx}, \varepsilon_{yy}, \varepsilon_{zz}, \gamma_{xy}, \gamma_{yz}, \gamma_{zx}] \end{array} \right\} \quad (2.54)$$

However, many 3-D structures can be made simpler under specific circumstances, and the state of stresses and strains can also be made simpler. As a result, 3-D elasticity problems can be reduced to 2-D elasticity problems and stress analysis. The 2-D finite element approach employs two main categories of models: plane-stress (Figure 2-21a) and plane-strain (Figure 2-21b). The 2-D plane-strain finite element elastic equations for the stress analysis of solids with linear elastic properties are described in this section.

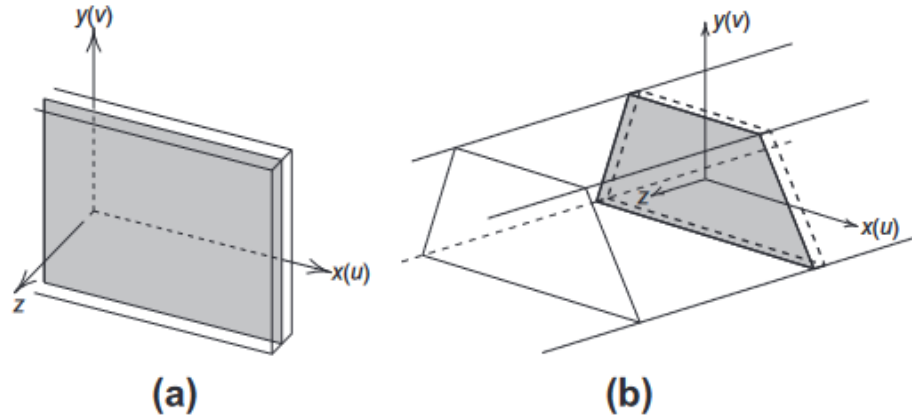


Figure 2-21. 2-D finite element analysis types: (a) plane stress, (b) plane strain (Zienkiewicz et al., 2013)

2.11.4.1 Plane-Strain Problems

If a solid body meets the assumptions listed below, it is considered to be in a state of plane-strain.

- (i) The structure is longer along one of the coordinate axes (called the z direction by convention), than it is in the x-y plane; its cross-sections are not considerably different along the body length; and the loads are mostly directed in the x-and/or y-direction with little variation along the z-axis. Tunnels, dams, and retaining walls are examples. See Figure 2-21(b).
- (ii) Any strain component that is connected to the z direction or normal to the x-y plane is presumed to be zero. These strain components can be mathematically defined as a state of loading and geometry and can be modelled as 2-D boundary value problems in the manner that,

$$\left. \begin{aligned} \epsilon_{zz} = \frac{\partial w}{\partial z} = 0, \quad \gamma_{xz} = \frac{\partial u}{\partial z} + \frac{\partial w}{\partial y} = 0, \quad \gamma_{yz} = \frac{\partial v}{\partial z} + \frac{\partial w}{\partial x} = 0 \\ \sigma_{xx} \neq 0, \sigma_{yy} \neq 0, \sigma_{zz} \neq 0, \tau_{xy} = \tau_{yx} \neq 0 \end{aligned} \right\} \quad (2.55)$$

- (iii) It is also assumed that the material of the body is linearly elastic, isotropic, and homogeneous.

The physical meaning of 2-D plane-strain problem is interpreted as the body is so long in the z-direction, only the Poisson effect can cause normal strain, which is so low or negligible. Additionally, it is assumed that only x-y-plane loadings are used, shearing stresses are likewise minimal and are often ignored (Hutton, D., 2003).

(a) Displacement field:

The displacement field is given in Cartesian coordinates by:

$$\mathbf{u}(\mathbf{x}, y, t) = \begin{bmatrix} u(\mathbf{x}, y, t) \\ v(\mathbf{x}, y, t) \end{bmatrix} \quad (2.56)$$

where positions are denoted by Cartesian coordinates x, y which can be written in matrix form as:

$$x = \begin{Bmatrix} x \\ y \end{Bmatrix} \quad (2.57)$$

(b) Strain matrix:

$$\varepsilon = [\varepsilon_{xx} \quad \varepsilon_{yy} \quad \gamma_{xy}]^T \quad (2.58)$$

Strain-displacement matrix:

For plane-strain case, the displacements are used to calculate the strains for a problem that is experiencing minor deformations, and the results can then be described in matrix form as below.

$$\varepsilon = \begin{Bmatrix} \varepsilon_{xx} \\ \varepsilon_{yy} \\ \gamma_{xy} \end{Bmatrix} = \begin{bmatrix} \frac{\partial}{\partial xx} & 0 \\ 0 & \frac{\partial}{\partial yy} \\ 0 & 0 \\ \frac{\partial}{\partial yy} & \frac{\partial}{\partial xx} \end{bmatrix} \begin{Bmatrix} u \\ v \end{Bmatrix} \quad \text{or} \quad \varepsilon = Su \quad (2.59)$$

In short form,

$$\varepsilon = Su \quad (2.60)$$

where: S is a matrix of differential operators, and u is the displacement field, given by:

The displacement matrix, $u = \{u, v\}^T$, in a plane element can be interpolated from nodal displacements (u_i, v_i) using shape (interpolation) functions, N_i , that are applied separately to each element, as follows:

$$\begin{Bmatrix} u \\ v \end{Bmatrix} = \begin{bmatrix} N_1 & 0 & N_2 & 0 & \dots \\ 0 & N_1 & 0 & N_2 & \dots \end{bmatrix} \begin{Bmatrix} u_1 \\ u_2 \\ v_1 \\ v_2 \\ \vdots \end{Bmatrix} \quad \text{or} \quad u = Nd \quad (2.61)$$

where:

N is the shape function matrix,

u is the displacement vector, and

d is the nodal displacement vector.

From the strain-displacement relationships, Eqs.2.60 and Eqs.2.61, the strain vector can be written in simplified form as

$$\varepsilon = Su = SNd, \quad \text{or} \quad \varepsilon = Bd \quad (2.62)$$

where $B = SN$ is the strain–displacement matrix.

(c) Stress matrix:

In plane-strain problems, only four components of stress are considered.

$$\sigma = [\sigma_{xx} \quad \sigma_{yy} \quad \sigma_{zz} \quad \tau_{xy}]^T \quad (2.63)$$

For plane-strain case, compared to the other stress components, the normal stress in the z-direction, σ_{zz} , although not zero, may be regarded as insignificant.

(d) Dynamic Equilibrium Equations:

The linear momentum or equilibrium equations for the 2-D behavior of a solid may be written in Cartesian coordinates as:

$$\frac{\partial \sigma_{xx}}{\partial x} + \frac{\partial \tau_{yx}}{\partial y} + f_{bx} = \rho \frac{\partial^2 u}{\partial t^2} \quad (2.64)$$

$$\frac{\partial \tau_{xy}}{\partial x} + \frac{\partial \sigma_{yy}}{\partial y} + f_{by} = \rho \frac{\partial^2 v}{\partial t^2} \quad (2.65)$$

Noting that $\tau_{xy} = \tau_{yx}$, the equilibrium equations may be written in a concise matrix form as

$$S^T \cdot \sigma + f_b = \rho \ddot{u} \quad (2.66)$$

where:

S is the differential operator which is the same as that given for strains in Eqs.2.59 and Eqs.2.60

f_b is the vector of external body forces in x-and-y directions, given as

$$f_b = [f_x \ f_y]^T \quad (2.66)$$

ρ is the mass density per unit volume and,

$\ddot{u} = \frac{\partial^2 u}{\partial t^2}$ is the acceleration vector.

(e) Constitutive Equations: Stress-Strain Relations

The constitutive equations for stress components for linear elastic isotropic materials can be expressed in matrix form as

$$\begin{Bmatrix} \sigma_{xx} \\ \sigma_{yy} \\ \tau_{xy} \end{Bmatrix} = \frac{E}{(1+\nu)(1-2\nu)} \begin{bmatrix} 1-\nu & \nu & 0 \\ \nu & 1-\nu & 0 \\ 0 & 0 & (1-2\nu)/2 \end{bmatrix} \begin{Bmatrix} \epsilon_{xx} \\ \epsilon_{yy} \\ \gamma_{xy} \end{Bmatrix} - \frac{E\alpha\Delta T}{1-2\nu} \begin{Bmatrix} 1 \\ 1 \\ 0 \end{Bmatrix} \quad (2.67)$$

which can be written in short form as,

$$\{\sigma\} = [D] \{\epsilon\} + \{\sigma_o\} \quad (2.68)$$

where

[D] = the stress-strain matrix (or the constitutive matrix),

$\{\sigma_o\}$ = the initial stress due to a temperature change (thermal loading), given by

$$\{\sigma_o\} = - \frac{E\alpha\Delta T}{1-2\nu} \begin{Bmatrix} 1 \\ 1 \\ 0 \end{Bmatrix} \quad (2.69)$$

Also,

$$\sigma_{zz} = \nu (\sigma_{xx} + \sigma_{yy}) - E\alpha\Delta T \quad (2.70)$$

The strain components can also be described in terms of stress as follows:

$$\begin{Bmatrix} \varepsilon_{xx} \\ \varepsilon_{yy} \\ \gamma_{xy} \end{Bmatrix} = \frac{(1+\nu)}{E} \begin{bmatrix} 1-\nu & -\nu & 0 \\ -\nu & 1-\nu & 0 \\ 0 & 0 & 2 \end{bmatrix} \begin{Bmatrix} \sigma_{xx} \\ \sigma_{yy} \\ \tau_{xy} \end{Bmatrix} + (1+\nu) \alpha\Delta T \begin{Bmatrix} 1 \\ 1 \\ 0 \end{Bmatrix} \quad (2.71)$$

The shear modulus or modulus of rigidity is computed by:

$$G = \frac{E}{2(1+\nu)} \quad (2.72)$$

(f) Stiffness Matrix

The Element stiffness matrix, k , describes the relationships of actions (applied force) and responses in terms of the DOFs for the set of nodes that form the element.

$$\{f\} = [k]\{u\} \quad (2.73)$$

The global stiffness matrix, K , is a symmetric matrix that is composed of the the sum of all the element stiffness matrices, and is given by,

$$\{F\} = [K]\{u\} \quad (2.74)$$

where: $\{f\}$ is the element load vector

$\{F\}$ is the equivalent global nodal load vector corresponding to the degrees of freedom

$[K]$ is the global structure stiffness matrix.

$\{d\}$ is the displacement vector consisting of global degrees of freedom

2.12 Numerical Solution of Linear Time-History Analysis

2.12.1 Numerical Solution Methods

FEA limits the number of degree of freedoms of the system to a finite number when solving a mathematical problem numerically (discretization). Fundamentally, this reduction is a discretization, which may involve in the time domain as well as the spatial domain analysis. A mathematical model is developed using partial differential equations in space and time for a particular physical model of the dam-foundation-reservoir system. With the inclusion of boundary conditions and, in the case of dynamic analysis, initial conditions for the system, the mathematical model represents the requirements of equilibrium conditions, constitutive relations, and continuity requirements (USBR, 2018).

For the numerical solution of the equations of motion for concrete gravity dams, there are two approaches, as per the guidelines provided in USACE 2003, EM 1110-2-6051:

- (i) **Direct integration method**, in which the equations of motion are solved through direct integration using a numerical step-by-step wise procedure, without making any transformation to the form of the equations.
- (ii) **Mode superposition method**, in which the equations of motion are solved in such a manner that they are first transformed into a more operative form (modal forms), and then they are solved either by a step-by-step integration in the time-domain, or by employing frequency-domain techniques.

In this study, the general descriptions of the direct integration method only and its procedures are briefly dicussed in the following sections. It is advised to look into other references or books on structural dynamics for details of the mode superposition method.

2.12.2 Time Domain Solutions: Equations of Motion

Time history analysis (also called transient dynamic analysis) involves determining the time history dynamic response of a structure to arbitrary time-dependent (varying) forces. With this kind of analysis, we may find out the time history responses of a structure to various combinations of transient or harmonic loads in terms of displacement, strain, stress, and force over time.

According to Chopra (2012), for a 2D idealized MDoF structural system subjected to an externally applied dynamic force $p(t)$, the equations of motion using using finite element formulation may be described in a matrix form by the following equation

$$M\ddot{u} + C\dot{u} + Ku = p(t) \quad (2.75)$$

with the physical meaning that implies:

Inertia forces + Damping forces + Elastic forces = Applied forces

where:

$p(t)$ = the time-varying external force or effective load vector, resulting from the earthquake ground shaking, includes inertia loads due to the mass of the structure and of the water.

f_s = the elastic resisting force

f_D = the damping resisting force

M = mass matrix

C = damping matrix

K = stiffness matrix

u = nodal displacement vector,

$\dot{u} = \frac{\partial u}{\partial t}$ = nodal velocity vector

$\ddot{u} = \frac{\partial^2 u}{\partial t^2}$ = nodal acceleration vector

As stated in USACE (2003) Engineer Manual EM 1110-2-6051, for concrete gravity dams, contributions from the interaction of the dam with the water and the foundation rock are included in the coefficient matrices and the effective load vector. The interaction effects of the dam, reservoir, and foundation are however treated differently depending on the finite-element formulation type chosen, as explained below.

- (i) In substructure finite-element formulation (discussed in section 2.9.2.1), the hydrodynamic forces, which are a component of the effective load vector, are typically used to represent the dam-reservoir interactions; and the dynamic stiffness (impedance matrix), which also incorporates interaction forces between dam and foundation, is used to represent the damping and stiffness.
- (ii) In standard finite-element formulation (discussed in section 2.9.2.2), with a massless foundation and incompressible water, the structure mass and the added mass of water are both contained in the global mass matrix, m , of the composite system. The stiffness matrix, k , represents the combined stiffness of the foundation and the structure. The inertia loads are caused by the mass of the structure as well as the added mass of water, and are included in the effective load vector, $p(t)$, arising from the earthquake ground shaking, as given by Eqs.2.42 in Section 2.10.1.1

2.12.2.1 Direct Integration Method

The solution process uses step-by-step integration procedures to provide an approximate solution of the equations of motion at ‘n’ discrete time intervals $0, \Delta t, 2\Delta t, 3\Delta t, \dots, t, t+\Delta t, \dots T$, where T is the duration of the input motion or loading and $\Delta t = T/n$ (USACE 2003, EM 1110-2-6051).

Numerous numerical integration methods have been developed to date. However, the methods that are frequently used in seismic analysis of concrete gravity dams for resolving the equations of motion (Eqs. 2.67) in the time domain are broadly classified into two:

- 1) Explicit method
- 2) Implicit method

In the continuous time domain, time is defined at discrete time points of $0, \Delta t, 2\Delta t, \dots, t$; the body is assumed to be in equilibrium; and the equilibrium is postulated in the “current” configuration $t+\Delta t$. Displacements are described as a function of $(t + \Delta t)$ as:

$$u(t + \Delta t) = u(t) + \Delta u \quad (2.76)$$

where Δu stands increments of displacements corresponding to time increment Δt

(a) Explicit Methods

Explicit techniques use the current state of a system to determine the state of the system at a future time. The fundamental concepts shared by the majority of explicit approaches include writing the equations of motion at the start of the time step, approximating the initial velocity and acceleration terms with finite difference expressions, and then solving for the response at the end of the time step.

$$u_{(t+\Delta t)} = u_t + \Delta t \cdot \dot{u}_t \quad (2.77)$$

Although explicit approaches are incredibly practical, they are only conditionally stable and will "blow up" if the time step is not sufficiently small. Explicit approaches are only stable if the time step size is less than a crucial one, which often relies on the structure's maximum natural frequency. Explicit approaches are frequently utilized for short-duration transient problems in structural dynamics due to the tiny time step required for stability considerations.

In the next section, the central difference method (CDM) as a very simple and most widely used finite difference method is briefly discussed. The details of other implicit methods can be obtained in reference books for numerical methods/computations for engineering applications.

(i) Central Difference Method

The central difference method (CDM) is a finite difference numerical method that is employed for differentiating and approximating the derivative of a function at a certain point by taking an average of the gradients of the function at that point's neighboring points.

In dynamic analysis of concrete dams, the method uses the following finite-difference expressions for approximation of the initial velocity and acceleration terms (USBR, 2018).

$$\ddot{u}_t = \frac{1}{\Delta t^2} (u_{t+\Delta t} - 2u_t + u_{t-\Delta t}) \quad (2.78)$$

$$\dot{u}_t = \frac{1}{2\Delta t} (u_{t+\Delta t} - u_{t-\Delta t}) \quad (2.79)$$

The displacement solution for time step $t+\Delta t$ (i.e. $u_{t+\Delta t}$) is obtained by considering the equations of motion (Eqs.2.75) at time step 't'

$$M\ddot{u}_t + C\dot{u}_t + Ku_t = p(t) \quad (2.80)$$

Substituting Eqs.(2.78 and 2.79) into Eqs.2.80, simplifies to:

$$\left(\frac{1}{\Delta t^2} M + \frac{1}{2\Delta t} C\right) u_{t+\Delta t} = p_t - \left(K - \frac{2}{\Delta t^2} M\right) u_t - \left(\frac{1}{\Delta t^2} M - \frac{1}{2\Delta t} C\right) u_{t-\Delta t} \quad (2.81)$$

As can be seen in the above equation (Eqs.2.71), the solution of $u_{t+\Delta t}$ is calculated from the solution of the equilibrium conditions at time t and the previous time $t-\Delta t$. This agrees with the definition on the explicit method (USBR, 2018). This equation shows that the stiffness matrix does not appear in the coefficient of $u_{t+\Delta t}$, and thus factorization of the (effective) stiffness matrix will not be required. If

mass and damping matrices are diagonal, even the matrices need not be assembled. In that case the solution is obtained at the element level which allows solving very large structural systems without substantial computer resources. These advantages make the central difference method convenient and efficient. However, the method is only conditionally stable and its stability depends on the choice of time step Δt . To obtain a stable solution, Δt must be $< T_n / \pi$, where T_n is the shortest natural period of the structural system. If the central difference method is used with a $\Delta t > T_n / \pi$ the solution will increase exponentially. This may not be a limiting factor for a SDOF system, because more than π or 3 steps are needed to adequately define one vibration cycle. However, the size of time step would be an issue for the MDOF systems where very short-period modes of vibration (compared with time step of input loading) are essential to the dynamic response of the system. In these situations extremely small time step would make the explicit methods undesirable (USACE 2003, EM 1110-2-6051).

(b) Implicit Methods

With an implicit algorithm the new state of the system cannot be calculated directly from the previous state; a coupled system of equations must be solved. In an implicit method the expressions for new values at $t+\Delta t$ use equilibrium equations at $t+\Delta t$, and thus include one or more values pertaining to that same step. This requires numerical solution schemes using iterative process for solving non-linear equations, typically the Newton-Raphson method, Newmark- β Method, and others. Implicit methods are usually unconditionally stable which means that different time step sizes can be chosen without any limitations originating from the method itself (USACE 2003, EM 1110-2-6051; USBR, 2018).

The generic form for the implicit method is as follow (USBR, 2018):

$$u(t + \Delta t) = u(t) + f(u(t + \Delta t), t + \Delta t) * \Delta t \quad (2.82)$$

In below section, the Newton-Raphson method and Newmark integration methods as the commonly used and powerful implicit methods for solving a system of nonlinear equations are briefly discussed.

(i) The Newton-Raphson Method

The Newton-Raphson method is based on the principle that if the initial guess of the root of $f(x)=0$ is at x_i , then if one draws the tangent to the curve at $f(x_i)$, the point x_{i+1} where the tangent crosses the x - axis is an improved estimate of the root as illustrated in Figure 2-22. Using the definition of the slope of a function, at $x=x_i$, the formula for solving a nonlinear equation of the form $f(x)=0$ is given by

$$x_{i+1} = x_i - \frac{f(x_i)}{f'(x_i)} \quad (2.83)$$

The Newton-Raphson algorithm follows the steps below to find the root of an equation of $f(x)=0$.

- 1) Compute the first derivative of $f(x)$, i.e determine $f'(x)$
- 2) Put an initial guess of the root, x_i , to estimate the new (improved) value of the root, x_{i+1} , as

$$x_{i+1} = x_i - \frac{f(x_i)}{f'(x_i)}$$

3) Compute the absolute relative approximate error, ε , as given by:

$$|\varepsilon_c| = \left| \frac{x_{i+1} - x_i}{x_{i+1}} \right| * 100 \quad (2.84)$$

4) Finally, convergence check is validation check that is done at the end of every guess to confirm whether or not the “guess” value is approaching the true solution. This is carried out by comparing the computed relative error in step (3) with the specified tolerance of the relative error, ε_s . If $|\varepsilon_c| > \varepsilon_s$, then go to Step (2), else stop the algorithm. Also, check if the number of iterations has exceeded the maximum number of iterations allowed. If so, one needs to terminate the algorithm and notify the user.

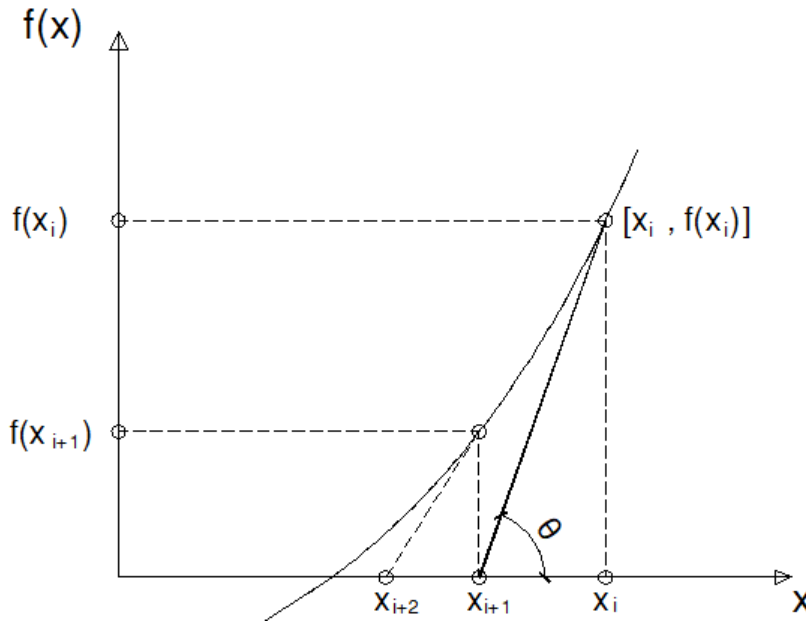


Figure 2-22 Schematic illustration of the Newton-Raphson method

(ii) Newmark Integration Method

The Newmark Integration method is a general step-by-step procedure with the following integration equations for the displacement and velocity at time step $t+\Delta t$ (Clough and Penzien 1993, Bathe and Wilson 1976, as cited in USACE 2003, EM 1110-2-6051).

$$\mathbf{u}_{t+\Delta t} = \mathbf{u}_t + \Delta t \dot{\mathbf{u}}_t + \Delta t^2 \left[\left(\frac{1}{2} - \beta \right) \ddot{\mathbf{u}}_t + \beta \ddot{\mathbf{u}}_{t+\Delta t} \right] \quad (2.85)$$

$$\dot{\mathbf{u}}_{t+\Delta t} = \dot{\mathbf{u}}_t + \Delta t [(1 - \gamma) \ddot{\mathbf{u}}_t + \gamma \ddot{\mathbf{u}}_{t+\Delta t}]$$

where β and γ are weighting factors and can be chosen to obtain optimum stability and accuracy.

This method is the default implicit method of analysis being applied in the Ansys mechanical (2023) program.

3 ANALYSIS INPUTS, STRUCTURAL MODELING AND SEISMIC PERFORMANCE EVALUATION

This chapter presents the various analysis inputs, materials used, structural modelling techniques followed, and the proposed methods/guidelines applied for comparative seismic performance evaluation of the existing Gibe III RCC dam and an idealized CVC dam. The methods followed, materials used, and modeling techniques applied in this study comprise description of the study area; collection, review and processing of data relating to the research; defining design/evaluation earthquakes for seismic response evaluation; development of finite element modeling and analysis procedures; the application of the US Army Corps of Engineers' (USACE) guidance for seismic performance evaluation of existing concrete gravity dams, as presented in the Engineer Manual EM1110-2-6053 (2007), and other relevant series of USACE publications for similar applications. .

The requirements to evaluate the two types of concrete gravity dams (RCC and CVC) has utilized the linear-elastic time history analysis procedures to determine a predictable performance for specified levels of seismic hazard. The seismic input and performance levels associated with serviceability, damage control, and collapse prevention were defined. The analysis and evaluation procedures and acceptance criteria for each performance level were described. The overall procedures consists of the linear-elastic time history analyses for estimation of seismic response and acceptance criteria that use demand-capacity ratios, damage control thresholds, and irrecoverable level of movements and post-earthquake stability condition to assess dam safety.

3.1 Overview of Research Methods and Materials

The overall research process of comparative seismic performance evaluation of RCC and CVC dams (with the case study of Gibe III dam) followed the following methodologies and materials:

- 1) Description of the study area
- 2) Collection, review and processing of data for the existing RCC dam
- 3) Establish design data for an idealized CVC dam
- 4) Establishment of earthquake design criteria
- 5) Development of design earthquakes and characterization of earthquake ground motions,
- 6) Establishment of analysis procedures,
- 7) Development of structural analysis models,
- 8) Prediction of earthquake responses of the structure, and
- 9) Interpretation and evaluation of the results.

The detail descriptions of each step, including the materials, approaches, guidelines, assumptions are discussed in the following sections.

3.2 Description of the Study Area

Gibe III hydropower plant is the third stage of the Gibe-Omo hydroelectric cascade which includes Gibe I and Gibe II (both in operation) and Koysha (currently under construction).

The project was initiated by the Federal Government of Ethiopian to develop the hydropower potential of the Omo/Gibe river with an objective to explore the hydropower power potential and improve substantially the power supply of the country. The dam has a storage capacity of 11.75 billion m³, and installed power generating capacity of 1870MW (6500 GWh/yr). It is currently operational.

3.2.1 Location of the Study Area

The Gibe III dam is a 243m high RCC dam located on the Omo River (UTM zone 37N at coordinates of 312,293 Easting and 757,225 Northing, or 6°50'50"N and 37°18'5"E on latitude/longitude map).

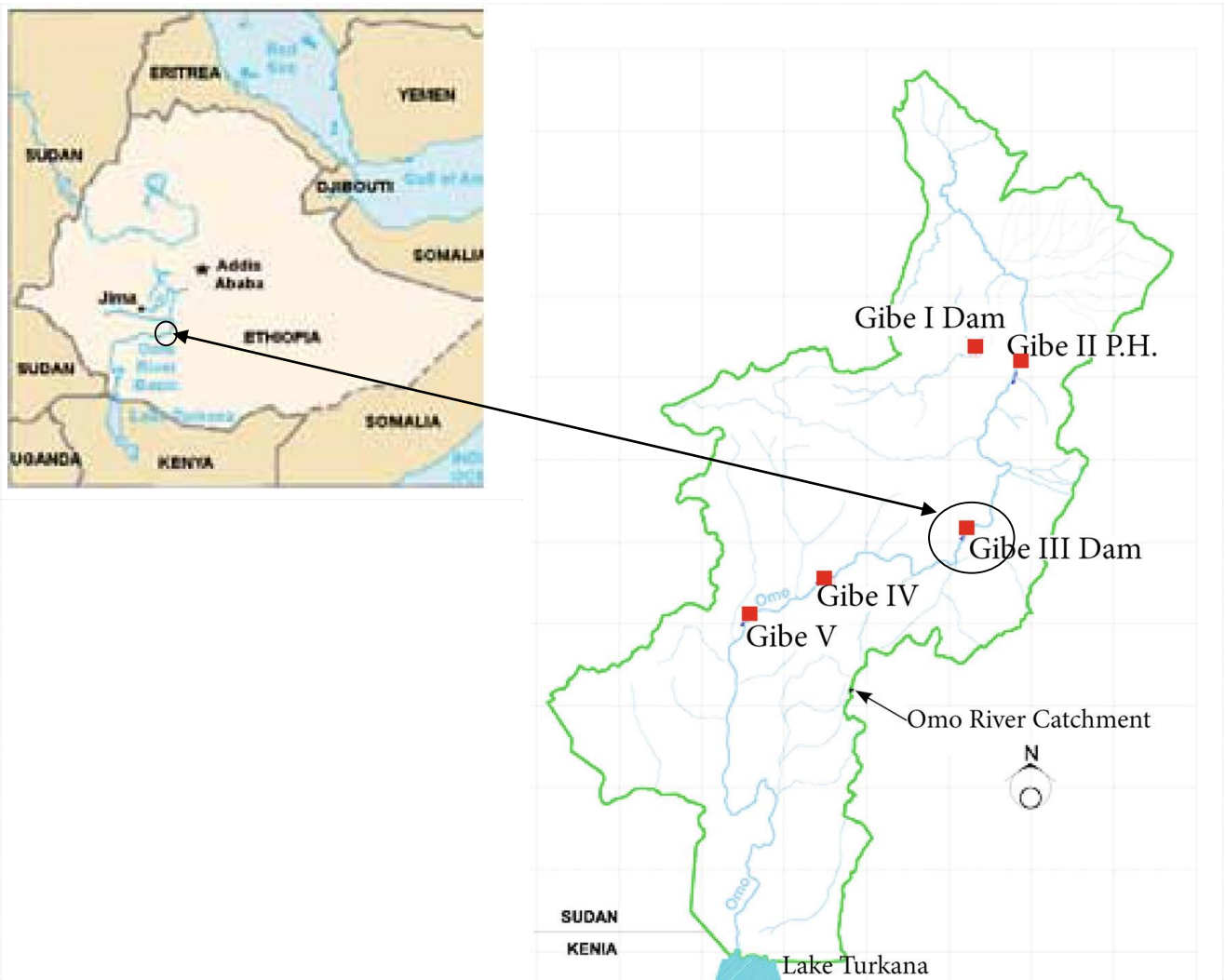


Figure 3-1 Location map of the ‘Gibe cascade’ of hydroelectric projects along the Omo River.
[Source: EEP Co (2007)]

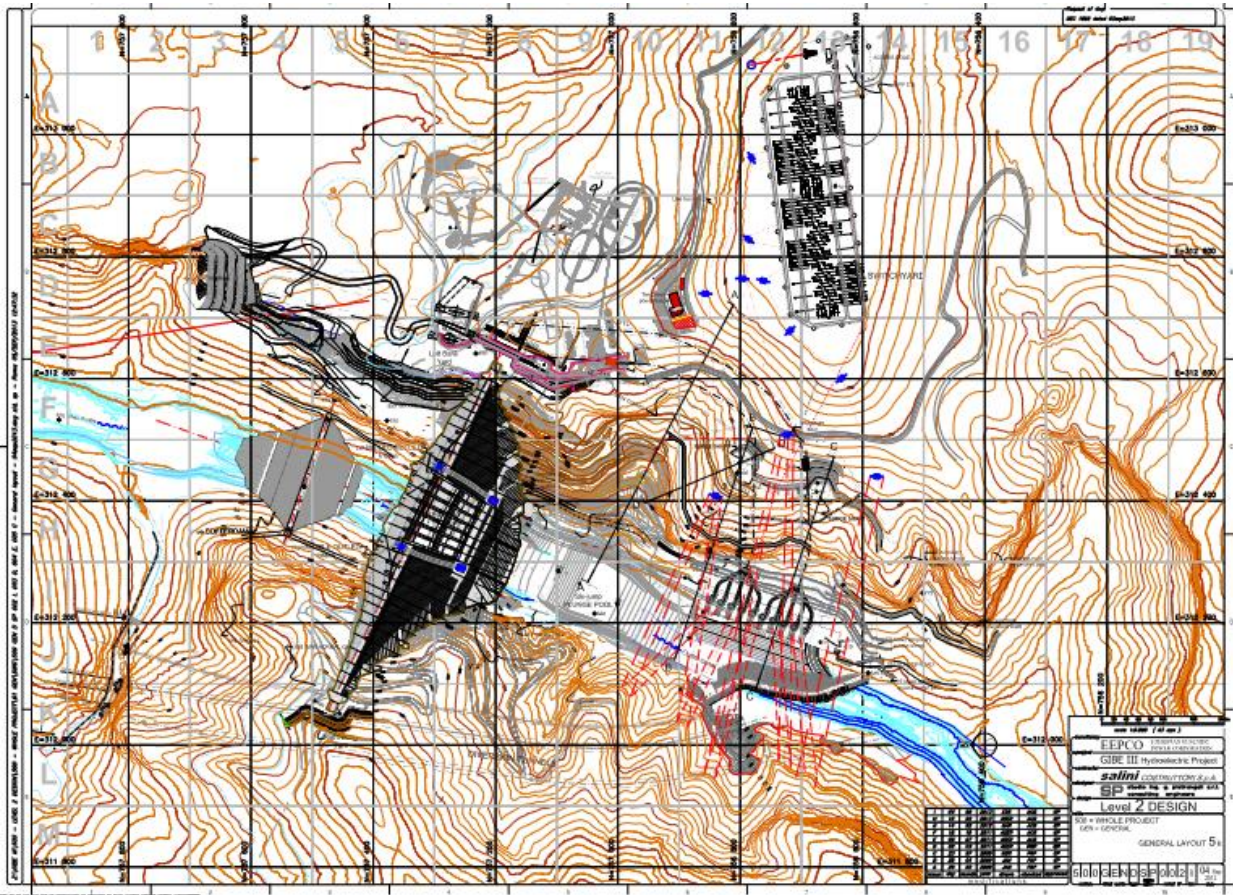


Figure 3-2 General layout of Gibe III dam [EEPCo, 2013]

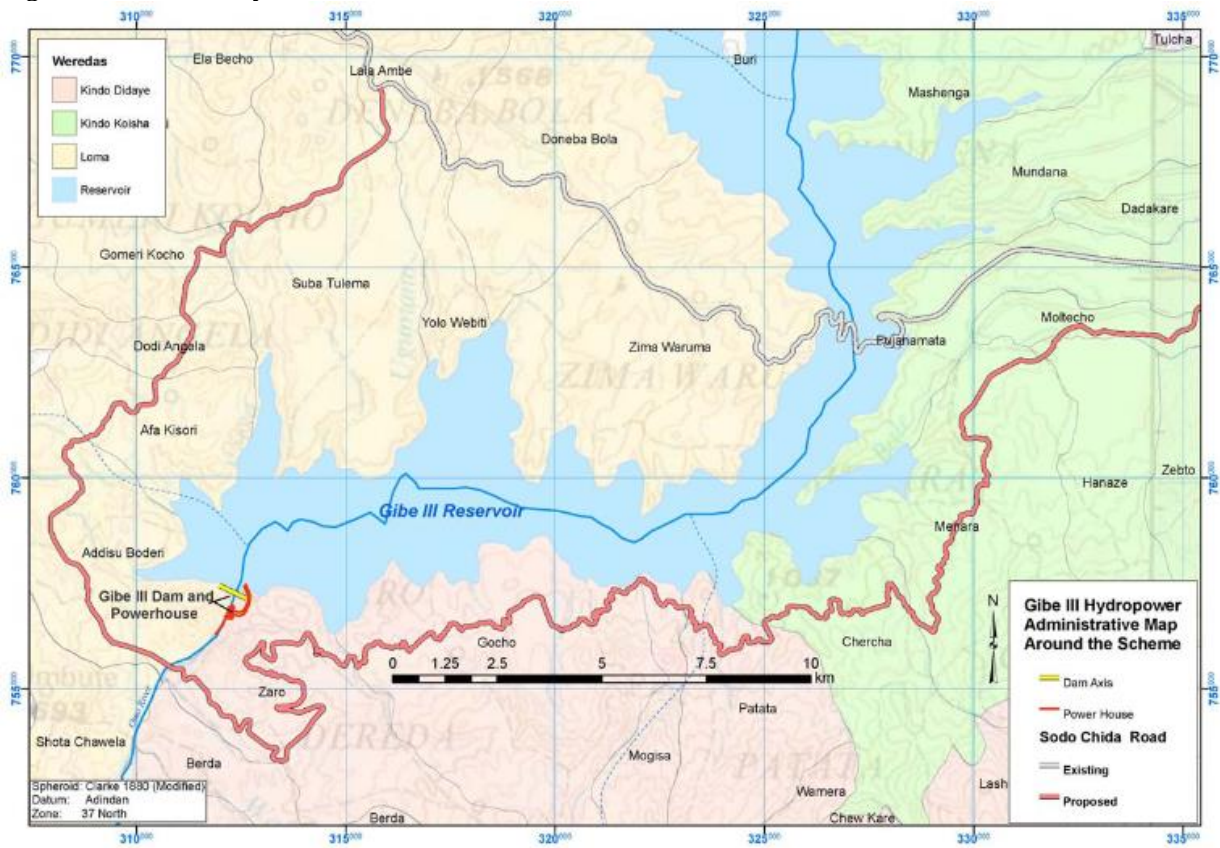


Figure 3-3 Gibe III reservoir map (EEPCo, 2013)

3.2.2 Salient Features of the Existing Gibe III RCC Dam

The main features of Gibe III hydroelectric plant are summarized in Table 3-1 below.

Table 3-1 Gibe III hydroelectric plant main features

1. Main dam body and accessories	
Name of dam	Gibe III
Location	Southern regional state, Ethiopia
Name of river	Omo
Type of structure	RCC
Foundation level (lowest)	650 masl (maximum overflow section)
Maximum height	243m (maximum overflow section) 236m (maximum non-overflow section)
Length of dam	620m
Spillways	7 in number, each with discharge capacity of 10600 m ³ /s
2. Reservoir	
Normal (maximum) operating water level	892 masl
Reservoir storage capacity (active)	11.75 x 10 ⁹ m ³ .
Standard Project Flood (SPF) level - 10600 m ³ /s	887.5 masl
Probable Maximum Flood (PMF) level-18600m ³ /s	893 masl
Minimum operating level	800 masl
Full reservoir level (sill)	875 masl (top of spillway)
Dam crest (non-overflow section)	896 masl
Maximum (flood) tailwater	695 masl
Silt elevation after 50 years	689masl
3. Tailwater	
Normal operating level	681 masl
Minimum tailwater level	677 masl
Standard Project Flood (SPF) level - 10600 m ³ /s	695 masl
Probable Maximum Flood (PMF) level-18600m ³ /s	701 masl
4. Hydroelectric plant/facilities	
Headrace tunnel	2 in number, length 1.0 km, diameter 11m
Power house	Outdoor; 10 Francis turbines, installed power 1870 MW.
Transmission lines	Length 3km, 400 kV

[Source: EEPCo, 2013]

3.2.3 Seismicity of the Study Area

The Main Ethiopian Rift (MER), the most seismically and tectonically active portion of the East African Rift System (EARS), lies close to the Gibe III dam. As evidenced by the damaging, large-magnitude earthquakes that have been recorded in the past, this region is capable of producing similar earthquake events. As a result, when designing new dams and assessing the safety of existing dams, the seismic danger must be taken into account (Aman, A., 2020).

3.2.3.1 Seismic Catalog for the Study Area: USGS (2023)

The historical and instrumentally recorded earthquake events that occurred within the period 1906 – 2021, as recorded in the Earthquake Catalog of the United States Geological Survey (USGS), indicated that there are several earthquake events with magnitudes in the order of M 5 to 7 that occurred within an epicentral distance of 200km from the dam site in different years (See table 3-5 below).

Table 3-2 Earthquake events within epicenters less than 200km from Gibe III dam site

S/ No.	Date/Time	Latitude	Longitude	Depth (km)	Mag. (USGS)	Epicentral distance from dam site (km)	Place
1	1906-08-25	6.96	38.34	10	5.99	115	14 km NW of Leku
2	1919-06-30	5.59	36.724	15	6.43	161	10 km SE of Jinka
3	1928-10-04	6.44	37.574	15	6.15	54	44 km N of Arba Minch
4	1944-09-06	5.92	38.043	10	6.04	132	38 km NW of Hagere Maryam
5	1954-06-30	5.69	37.145	15	6.02	130	24 km W of Gidole
6	1968-01-23	8.6	37.619	15	5.19	198	38 km WNW of Waliso
7	1983-12-02	7.03	38.599	10	5.1	145	14 km ESE of Hawassa
8	1985-08-20	5.52	36.143	10	5.4	195	55 km WSW of Bako
9	1987-10-07	6.22	37.814	10	5.3	90	35 km NE of Arba Minch
10	1987-10-07	6.35	35.88	10	5.3	166	78 km SSE of Mizan Teferi
11	1987-10-25	5.41	36.751	12	6.3	171	28 km SSE of Jinka
12	1987-10-28	5.74	36.73	10	6.0	138	13 km NE of Jinka
13	1993-09-18	5.35	37.561	10	5.2	169	13 km NE of Konso
14	1995-01-20	7.16	38.441	13.8	5.0	130	11 km NNW of Hawassa
15	2010-12-19	7.52	37.839	10	5.1	95	3 km SSW of Hosaina
16	2011-03-19	6.66	38.521	10	5.0	136	13 km ENE of Wendo
17	2017-01-27	7.67	38.668	10	5.3	176	29 km S of Zeway

[Source: USGS, 2023; website: <https://earthquake.usgs.gov/earthquakes/search/>]

3.3 Collection, Review and Processing of Data for the Existing Dam

To facilitate the successful execution and completion of this research work, valuable data which are found highly relevant to the study were collected from two sources: the dam owner and previous researches relevant to the study area. :

(a) Data collected from project office of the dam owner.

The Gibe III hydroelectric plant is owned and operated by the state-owned organization known as Ethiopian Electric Power (EEP), formerly called Ethiopian Electric Power Corporation (EEPCo). The project data collected from EEP mainly include technical reports, dam structural drawings, material properties determined through trial mixes and laboratory testing, which were used in the original design, quality control and implementation of the Gibe III RCC gravity dam project.

The technical reports and relevant data collected from EEP specifically included the following:

- (i) Plans and drawings (EEPCo, 2013/2016)
 - Location plans, topographic map, geological/geotechnical profiles
 - Geometric details: dam cross-sections at maximum overflow and non-overflow sections
 - RCC zones:
- (ii) Hydrological data: estimated maximum and minimum operating levels of reservoir and tailwater levels, silt (sediment) deposits.
- (iii) Seismic hazard assessment for Gibe III hydroelectric project (EEPCo, 2007);
- (iv) Geological/geotechnical study for Gibe III hydroelectric project (EEPCo, 2008);
- (v) RCC mixes, tests, and properties (EEPCo, 2011)
- (vi) RCC zoning, thermal properties (Studio Pietrangeli, 2016)

(b) Data collected from previous researches relevant to the study area

- (i) Site-specific seismic hazard study for Gibe III dam site (Aman, A., 2020)
- (ii) Regional seismic hazard study for Ethiopia (Ayele, A., 2017)

The details of the relevant information for the existing RCC dam collected from EEPCo and previous seismic studies relevant to the study area, which are deemed necessary as inputs for this study, and the researcher's reviews and recommendations, are discussed below in below sections 3.3.1 through to 3.3.5

3.3.1 Geometric Data

The typical cross-sections of Gibe III RCC dam at maximum overflow and non-overflow sections as obtained from the dam owner's project office and used in the current study are shown in Figure 3-4 (a) and (b) below respectively.

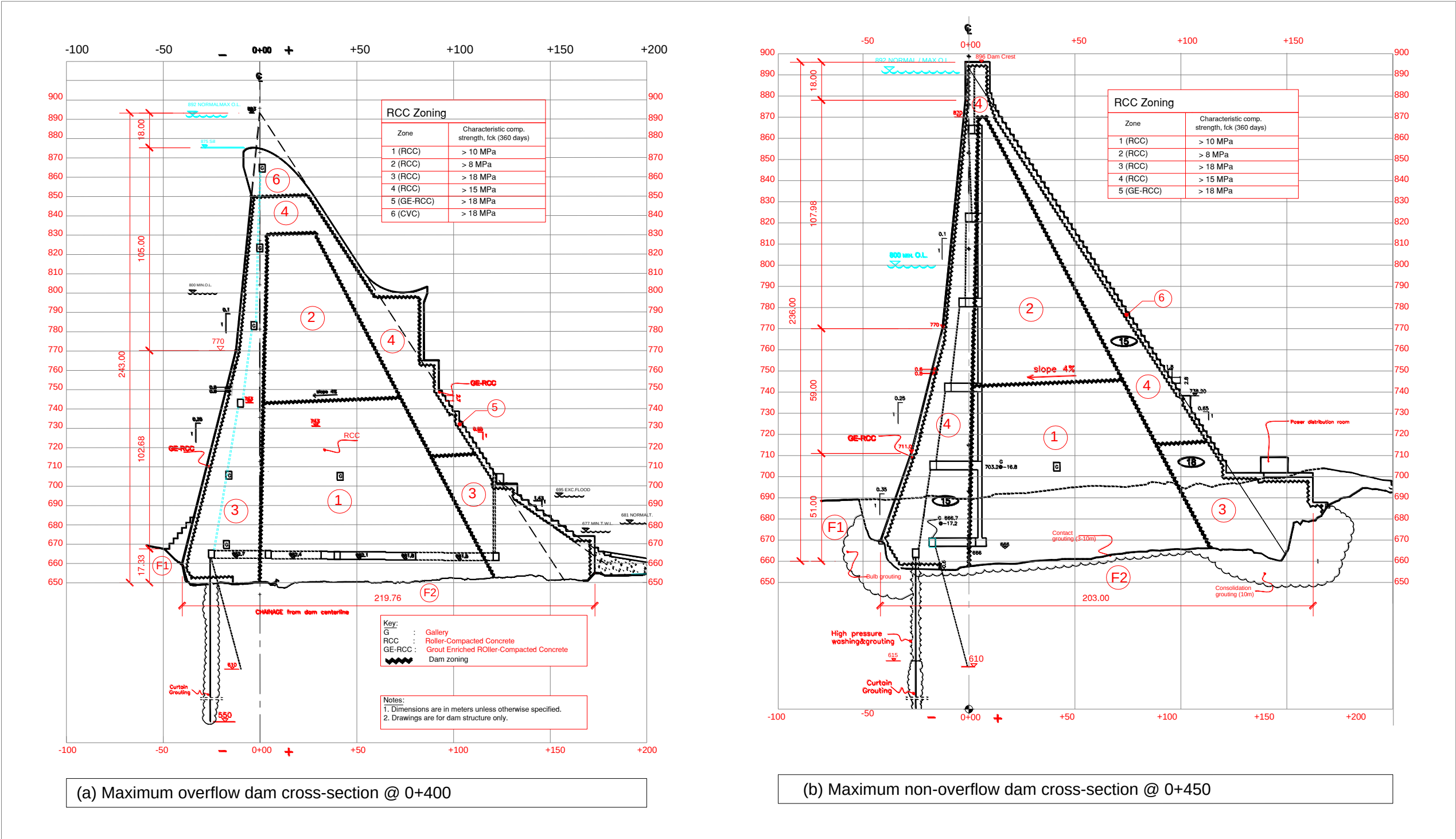


Figure 3-4 Gibe III dam cross-sectional drawings (a) maximum overflow dam section, and (b) maximum non-overflow section [EEPCo (2015)]

3.3.2 Geological/Geotechnical Data

According to EEPCo study (2008), the geological/geotechnical assessment carried out at Gibe III dam site included exploratory / diversion tunnel excavations, borehole drilling, in-situ tests, laboratory tests, inspection tunnels, petrographic analysis, and dilatometric tests. The geological/geotechnical study results and recommendations are briefly summarized as follows.

3.3.2.1 Geological Settings and Dam Foundation Zones

The Gibe III dam foundations are characterized by the presence of a sequence of volcanic and volcano-sedimentary rocks at the base of Omo valley with varying thicknesses of the strata at different positions along the proposed dam axis.. The following geological settings at the valley floor, where the maximum overflow and non-overflow sections are placed, were considered in this study:

Layer-1 [SW-T]: This is the top layer that is mainly composed of Slightly-to-Moderately Weathered Trachyte that exists as outcrops in the valley floor (thickness 10m)

Layer-2 [U-T]: The underlying strata of Unweathered Trachyte that extends deep (upto 200m) below the dam foundations on the left abutment, and also outcrops at the valley floor. .

3.3.2.2 Foundation Rockmass Properties

The following elastic properties were established for the foundation rockmass based on the results of the tests conducted at the project by EEPCo (2011) and were adopted in this study.

(a) Elastic Properties

Table 3-3 Rockmass foundation elastic properties (EEPCo, 2011)

Parameter	Design- ation	Unit	Rock mass layers	
			Layer-1: SW-T [Slightly Weathered Trachyte], Disturbed rockmass, D= 0.5	Layer-2: U-T [Unweathered Trachyte], Undisturbed rockmass, D= 0
Deformation Modulus	E_d	MPa	22000	32000
Poisson's ratio	ν	-	0.15	0.15

[Source: EEPCo, 2011]

*The deformation modulus of Layer-1 at slightly disturbed state (i.e 22000 MPa) and that of Layer-2 at undisturbed state (i.e. 32000 MPa) were considered in this study..

(b) Mass Density

Massless foundation model was considered in the study, and thus zero density was assumed for both layers of the foundation rockmass.

(c) Dynamic Properties

The dynamic properties of the rockmass foundations - deformation modulus and Poisson's ratios - are assumed to be the same as the values of static properties specified in Table 3-3 for similar parts of the structural system.

3.3.2.3 Silt (Sediment) Deposits

Silt deposits with the following geotechnical characteristics were considered in the study (EEPCo, 2008):

- Angle of internal friction, $\phi' = 25$ degrees
- Cohesion, $C' = 0$ KN/m²
- Saturated unit weight, $\gamma_{\text{sat}} = 17$ KN/m³

In addition to the above properties, the following properties were assumed for silt/sand deposit as a continuum soil to employ a linear-elastic material model (Stromblad, N., 2014).

- Young's modulus: 10 – 25 MPa (for loose sand); 50-81 MPa (for dense sand)
- Poisson's ratio: 0.20 – 0.35 (for loose sand); 0.30 – 0.40 (for dense sand).

In this study, Young's modulus of 50MPa and Poisson's ratio of 0.40 were taken.

3.3.2.4 Rock mass foundation – concrete interface

The rockmass-concrete interface shear strength was estimated for the project design stage based on in-situ tests (EEPCo, 2008), and the same parameters were adopted in this study

Cohesion: $c' = 0.3$ MPa and

Angle of internal friction: $\phi' = 50$ degrees.

The tensile strength of the interface was assumed zero since it is mainly controlled by the tensile strength of the rock mass immediately below the concrete-rock contact.

3.3.3 Hydrological Data

According to EEPCo (2013/2016) the hydrological data established for the project design stage comprises important information including the estimated maximum and minimum operating levels of reservoir and tailwater for the estimated floods of various return periods, and also silt (sediment) deposits after 50 years, as presented in Table 3-1 in Section 3.2.2.

3.3.4 Previous Seismic Studies Relevant to the Study Area

Site-specific seismic hazard studies were conducted by the dam owner (EEPCo, 2007) for the design stage of the project, and also by Aman, A. (2020) for his PhD dissertation. Furthermore, regional seismic study was conducted by Atalay, A.(2017). These studies were found relevant and useful in this study and briefly presented below along with the researcher's comparative reviews and recommendations for usage in this study.

(a) Seismic Hazard Assessment for the Design Stage of Gibe III Project (EEPCo, 2007)

According to EEPCo study (2007), the seismic hazard assessment for the Gibe III dam was made to estimate design earthquakes - Maximum Credible Earthquake (MCE), Maximum Design Earthquake (MDE), and Operating basis Earthquake (OBE), as recommended by ICOLD method (1989) by considering seven (7) seismogenic faults identified around or nearby the dam location. The horizontal PGAs were then derived for each of the critical faults using a Deterministic Seismic Hazard Assessment (DSHA) method. Accordingly, the estimated design earthquake ground motions of MCE, MDE and OBE used for the dynamic analysis and stability analysis in the design stage of the project are summarized in Table 3-4 below:

Table 3-4 Estimated earthquake ground motion for the design of Gibe III dam project (EEPCo, 2007)

Design Earthquake	Average Return Period (T_R)	Probability of Exceedance (P_E)	Service life (T_L)	Damp-ing ratio	Method of estimation	Estimated magnitude of PGA	Duration of signal (sec.)	Ground condition
MCE	--	-	-	5%	DSHA	0.15g	16	Horizontal bedrock site $V_{s30} = 760$ m/s
MDE	950 yrs	10%	100	5%	DSHA	0.11g	11	
OBE	145 yrs	50%	100	5%	DSHA	0.06g	7	

[Source: EEPCo, 2007]

(b) Site-Specific Seismic Hazard Study for Gibe III Dam Site (Aman, A., 2020)

According to the study by Aman, A. (2020), the site-specific seismic hazard assessment for Gibe III dam site was undertaken using Probabilistic Seismic Hazard Assessment (PSHA) approach as per ICOLD (2016) recommendations for large storage dams and the author estimated earthquake hazards in terms of PGAs for different return periods as summarized in Table 3-5 below.

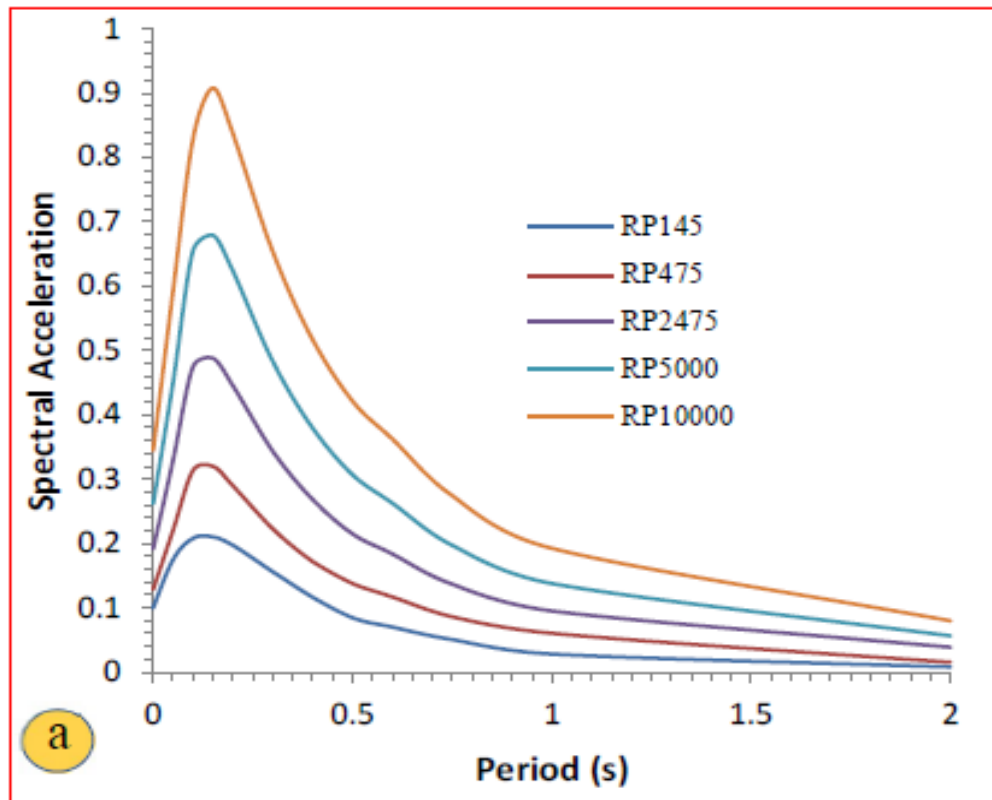
Table 3-5 Estimated site-specific seismic hazards for Gibe III dam (Aman, A., 2020)

Design Earthquake	Average Return Period (T_R)	Probability of Exceedance (P_E)	Service life (T_L)	Damping ratio	Method of analysis	Estimated magnitude of PGA	Ground condition
SEE	10000yrs	-	-	5%	PSHA	0.34g	Average rock site $V_{s30} = 760$ m/s
OBE	145yrs	50%	100	5%	PSHA	0.10g	

[Source: Aman, A., 2020]

Site-Specific Response Spectra for Gibe III Dam Site (Aman, A., 2020):

Aman, A. (2020) conducted site-specific seismic hazard analysis of Gibe III dam site using the probabilistic seismic hazard analysis (PSHA) Accordingly, he estimated the PGA-values for the SEE (10,000 years return period) as 0.34g for the horizontal earthquake component on the outcropping rock with 5% damping. In addition, the PGAs for other return periods (145, 475, 2475 and 5000 years) were estimated. Uniform hazard spectra (UHS) showing plots of spectral accelerations versus the natural periods of vibration of single degree of freedom (SDOF) systems at various return periods for horizontal earthquake component on generic rock at 5% damping are shown in Figure 3-5 below.



[key: RP = return period]

Figure 3-5 Uniform hazard spectra of Gibe III dam site (Aman, A., 2020)

(c) Regional Seismic Hazard Study for Ethiopia and Horn Africa (Ayele, A., 2017)

Ayele, A. (2017) conducted seismic hazard study using PSHA method and produced seismic hazard map (Figure 3-6) for the horn of Africa regions. Based on this seismic hazard map, the PGAs of the dam site for 475-year and 2475-year return period earthquake event in generic rock site could be estimated as presented in Table 3-6.

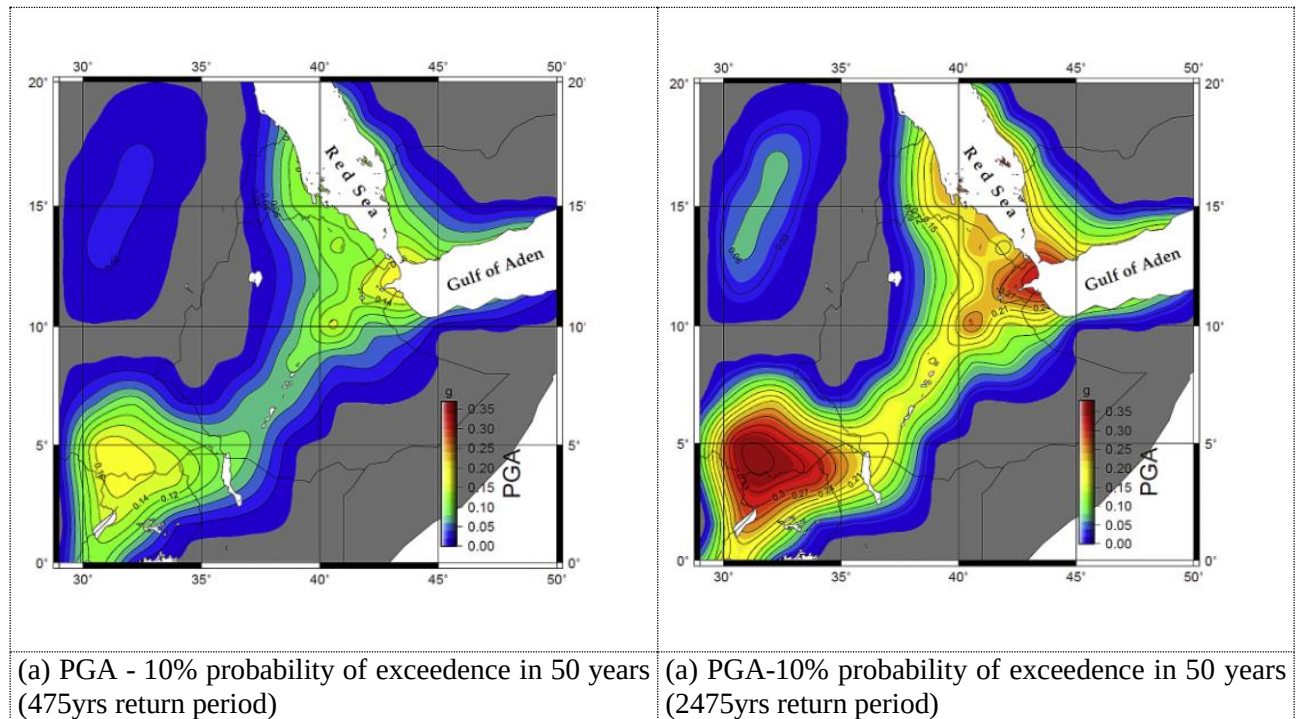


Figure 3-6 Seismic hazard map for horn Africa regions (Ayele, A., 2017)

Table 3-6 Estimated seismic hazard levels for Gibe III dam site (Ayele, A., 2017)

Average Return Period (T_R)	Probability of Exceedence (P_E)	Service life (T_L)	Damping	Method of analysis	Estimated PGA	Ground condition
475	10%	50	5%	PSHA	0.10g	Generic rock site, $V_{s30} = 760$ m/s
2475	2%	50	5%	PSHA	0.18g	

(d) Recommended Earthquake Ground Motion Demands for this Study

To select the future ground motion demands, the hazard levels and PGAs determined by EEPCo (2007) using DSHA in paragraph (a) were compared with the seismic hazard study results and PGAs recommended by other researchers in paragraphs (b) and (c) above, and accordingly the following findings and recommendations were drawn by the researcher:

- (i) The PGA of 0.15g for MCE used for the design stage of the Gibe III hydroelectric project by EEPCo (2007) is significantly lower than the PGA of 0.34g determined using PSHA method for the SEE (10000-year return period) by Aman, A. (2020). It is even lower than the PGA of 0.18g that could be estimated for 2475 year-return period for a region near the dam site according to the seismic hazard map for Ethiopia and horn of Africa regions produced by Ayele, A.(2017).
- (ii) Similarly, the PGA of 0.06g for OBE (145-year return period) used in the design stage of the Gibe III dam is much smaller than the PGA of 0.10g estimated for the corresponding OBE of the same return period by Aman, A.(2020).
- (iii) For this particular study, the PGAs estimated for SEE and OBE using PSHA method by Aman, A.(2020) were applied for developing the ground motion parameters and acceleration-time histories for dynamic analysis.

3.3.5 RCC Zoning and Properties

According to the EEPKO (2011) report entitled “RCC Full-Scale Trials: Level-2 Design,” prepared by the design-build contractor (Salini Costruttori /builder/, Studio Pietrangeli /designer/), the design parameters of the RCC mixes, tests and properties adopted in the design stage of the project and also utilized in this study are briefly summarized as follows:

3.3.5.1 Static Properties

(a) Mix Design, Strengths and Elastic Properties

The entire RCC dam cross-sectional geometry at both overflow and non-overflow sections is divided into different zones having different concrete mixes, strengths and other properties as shown in Figure 3-4 in Section 3.3.1. The RCC zoning generally comprises zones with higher compressive strength requirements ($f_{ck} > 15$ MPa) for the exterior regions and zones with lower strength requirements ($f_{ck} > 8-10$ MPa) for the dam central regions to satisfy structural design and stability requirements.

Table 3-7 RCC zoning, mixes, strengths, and elastic properties

RCC Zone	Designation	RCC mixes/strength		Elastic properties		Mass Density, ρ_c	Remark
		Minimum cement content	Compressive strength, $f_{c'}$ (365 days)	Young's modulus, E_c	Poisson's ratio, ν		
		(kg/m ³)	(MPa)	(GPa)		(kg/m ³)	
1	RCC1	90	>10	15	0.20	2350	Both overflow and non-overflow sections have RCC zones 1 to 5.
2	RCC2	70	>8	13	0.20	2350	
3	RCC3	120	>18	20	0.20	2350	
4	RCC4	105	>15	18	0.20	2350	

RCC Zone	Designation	RCC mixes/strength		Elastic properties		Mass Density, ρ_c	Remark
		Minimum cement content	Compressive strength, f_c' (365 days)	Young's modulus, E_c	Poisson's ratio, ν		
		(kg/m3)	(MPa)	(GPa)		(kg/m3)	
5	GE-RCC	190	>18	20	0.20	2350	

[Source: EEPCo, 2011]

(b) Lift Joints Direct Tensile Strengths

(i) Study by the Designer/Engineer (Studio Pietrangeli, 2016)

Studio Piterangeli (2016), the designer/Engineer of the project, studied the relationship between the measured values of compressive and splitting tensile strength tests on samples of parent concrete mixes with cement contents of 70, 90, 105, and 120kg/m3 at various ages and presented the relationships in graphs shown in Figure 3-7 (a) and (b) below.

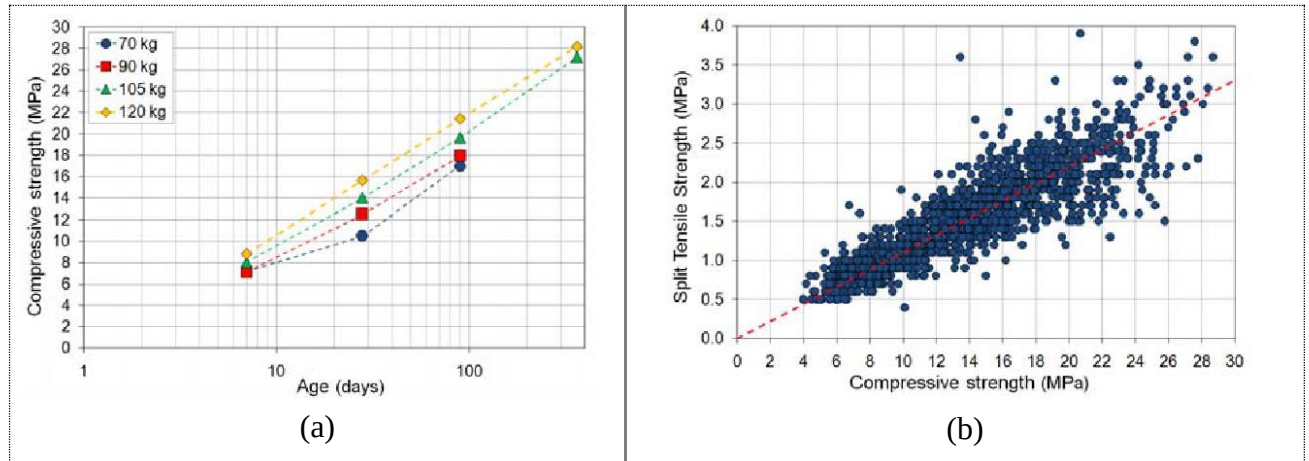


Figure 3-7 Splitting tensile strength to compressive strength relationship (Studio Pietrangeli, 2016)

The relationship between the compressive and splitting tensile strength was approximated by the following equation:

$$f_{st}' = 0.10 * f_c' \quad (3.1)$$

where: f_{st}' = split tensile strength (MPa)

f_c' = characteristic concrete compressive strength (MPa)

The direct tensile strengths for the RCC lift joints with bedding mortar were indirectly derived from indirect split-tension results by using the following relationship equation developed by Schrader (1995b) and Schrader et al. (2003a) (as cited in Nawy, E.G., 2008).

Direct tensile strength = $[0.3 \log(10 \times \text{compressive strength})] \times (\text{split tensile strength})$

$$f_t' = [0.3 \log(10 * f_c')] \times f_{st}' \quad (3.2)$$

where: f_t' = direct tensile strength (MPa)

(ii) USACE (2000) guidance:

Canon (1995), as cited in USACE 2000, EM 1110-2-2006, developed the following empirical relationships based on historical data to estimate RCC lift joints tensile strength from splitting tensile tests conducted on the parent RCC with one-year characteristics compressive strength less than 24.1MPa, nominal maximum size of aggregates not larger than 75mm, and mortar bedded lifts.

$$\text{Split tensile strength of parent concrete: } f_{st}' = 0.08f_c' \text{ to } 0.17 f_c' \quad (3.3)$$

$$\text{Direct lift joint tensile strength, } f_t' = 0.04f_c' \text{ to } 0.09f_c' \quad (3.4)$$

in which high values of lift joint direct tensile strength are based on crushed, high strength aggregates and bedded lifts.

(iii) Recommended RCC lift joints' direct tensile strength for this study

The estimates of RCC lift lines direct tensile strength were made by comparing estimates obtained from the above two empirical correlations (i) and (ii) as presented in Table 3-8 below. In Eqs.(3.3) and Eqs.(3.4), intermediate values, $f_{st}'=0.15f_c'$ and $f_t'=0.08f_c'$, were considered.

Table 3-8 RCC lift joints' direct tensile strengths

RCC zoning	Compressive strength (365 days) f_c' (MPa)	Studio Pietrangeli's study (2016)		USACE (2000) guidance	
		Split tensile strength of parent concrete (cores) f_{st}' (MPa)	Lift joint direct tensile Strength f_t' (MPa)	Split tensile strength of parent concrete (cores) f_{st}' (MPa)	Lift joint direct tensile strength f_t' (MPa)
RCC1	>10	1.00	0.60*	1.20	0.80
RCC2	>8	0.80	0.46*	0.96	0.64
RCC3	>18	1.80	1.22*	2.16	1.44
RCC4	>15	1.50	0.98*	1.80	1.20
RCC5(GE-RCC)	>18	1.80	1.22*	2.16	1.44

* The lift joints' direct tensile strength values estimated using the two relationships are very close to each other, and therefore the values estimated by the Designer/Engineer, Studio Pietrangeli (2016), were considered in this study.

(c) RCC Lift Joints' Shear Strength Parameters

The mechanical behaviors of the horizontal lift joints were studied by conducting trial tests to determine the influence of the RCC lift joints on the shear strength (cohesion and friction angle) at the horizontal joints of the RCC dam structure. The shear strength parameters (C' and ϕ') shown in table below were estimated for the different classes of concrete based on the one-year unconfined compressive strength, f_c' , of RCC determined from laboratory tests.

Table 3-9 RCC horizontal lift joints shear strength parameters

RCC zone	Characteristic compressive strength (365 days)	Internal friction angle	Cohesion
	fc' (MPa)	ϕ' (Deg.)	C (MPa)
1	>10	45	0.6
2	> 8	45	0.4
3	>18	45	1.2
4	>15	45	1.0
5	>18	45	1.2

[Source: EEPCo, 2011]

The shear strength parameters indicated in the above table were adopted in the study..

3.3.5.2 Thermal Properties

The thermal properties of the RCC mixes were determined through laboratory testing by the Designer/Engineer of the project (Studio Pietrangeli, 2016) and were directly adopted in this study as presented in Table 3-10 below.

Table 3-10 Thermal properties

Parameter	Unit	RCC mix / cement content (kg/m3)				
		70	90	105	120	200
Specific heat (c_h)	kJ/kg-°C	0.820	0.820	0.820	0.820	0.820
Thermal conductivity (k)	W/m-°C	0.890	0.890	0.890	0.890	0.890
Thermal diffusivity (h^2)	m ² /hr	0.0016	0.0016	0.0016	0.0016	0.0016
Coefficient of thermal expansion (α)	($\times 10^{-6}$) 1/°C	7.6	7.7	7.8	7.9	8.10
Adiabatic temperature rise (ΔT_{ad}) at the age of 365 days	°C	11.0	13.6	15.6	17.5	27.4
Represented RCC zone		2	1	4	3	5
Key: W = watt; kJ = kilo joule						

[Source: Studio Pietrangeli, 2016]

3.3.5.3 Dynamic Properties

The dynamic material properties used as inputs into a linear-elastic dynamic analysis of the RCC dam were estimated based on the USACE's (2007) guidelines for correlation between the static and dynamic properties (static properties multiplied by the conversion factor) as summarized in Table 3-11 below.

Table 3-11 RCC dynamic properties

Dynamic properties	Design-ation	Unit	Conversion factor	RCC zones				
				RCC1	RCC2	RCC3	RCC4	RCC5
Dynamic modulus	E_d	MPa	1.15	17250	14950	23000	20700	23000
Poisson's ratio	ν	..	0.70	0.14	0.14	0.14	0.14	0.14
Compressive strength	f_c'	MPa	1.15	11.5	9.2	20.70	17.25	20.7
Lift joints direct tensile strength	f_t'	MPa	1.50	0.9	0.70	1.83	1.47	1.83

As per USACE (2003) guidelines, for the foundation rockmass, the dynamic properties were considered similar to the elastic properties, as presented in Table 3-3 in Section 3.3.2.2

3.4 Developing Design/Evaluation Data for an Idealized CVC Dam

To perform comparative seismic performance evaluation of RCC and CVC dams, an idealized CVC dam of equivalent size should be considered.. The only difference between the two types of dams exists in the concrete properties while the foundation rockmass properties, static loads, seismic design considerations, and damping remain the same for both dam types. The entire CVC dam zone is considered to be isotropic and homogeneous with the required properties established for the comparative study in accordance with the project database (EEPCO, 2011) and with reference to the relevant USACE (2000) guidance for the purpose of this study.

3.4.1 Geometric Data

The proposed typical cross-sectional drawing for the non-overflow CVC dam section adopted for this study is presented in Figure 3-8 below.



The geological/geotechnical properties considered in the dynamic analysis of the idealized CVC dam are considered to be exactly similar to the the properties defined for the RCC dam in section 3.3.2.

The hydrological data considered in the dynamic analysis of the idealized CVC dam are considered to be exactly similar to the the data established for the RCC dam in section 3.3.3.

The previous seismic studies and recommendations discussed in Section 3.3.4 (a) to (d) were also applied in the case of the idealized CVC dam to be analyzed under the same seismic geological/tectonic setting.

3.4.5 Material Properties

3.4.5.1 Static Properties

(a) Mix Design, Strength and Elastic Properties

All concrete blocks of the idealized 2-D CVC gravity dam section were proposed to be isotropic and homogeneous concrete sections. The CVC properties were directly adopted from the properties of the existing CVC region provided for the top of the overflow (spillway) section of the existing RCC dam, as presented in EEPCo drawings (2011) and also shown in Figure 3-4 in section 3.3.1.

Table 3-12 CVC strength and elastic properties

Dam zone	Designation	Minimum cement content (kg/m ³)	Compressive Strength (365 days) fc' (MPa)	Elastic properties		Mass density ρ _c (kg/m ³)
				Young's modulus Ec (GPa)	Poisson's ratio ν	
1	CVC	200	>18	20	0.20	2350

[Source: EEPCo, 2011]

(b) Lift joints direct tensile strength (USACE 2000 guidance)

The CVC lift line's direct tensile strength was estimated using empirical correlations developed by Cannon (1995), as cited in USACE 2000, for conventional mass concrete having compressive strength less than or equal to 20.7MPa and nominal maximum size aggregates (NMSA) not greater than 75mm. Review of existing EEPCo material testing reports (EEPCo, 2011) indicated that the aggregates used for both RCC and CVC mixes have NMSA smaller than 75mm.

Split tensile strength of parent concrete: $f_{st}' = 0.10f_c'$ to $0.15f_c'$ (3.5)

Lift joints direct tensile strength: $f_t' = 0.055f_c'$ to $0.085f_c'$ (3.6)

Considering an intermediate strength values in Eqs.(3.5) and (3.6) and the use of crushed, high strength aggregates and mortar bedded lifts, the lift joint direct tensile strength was estimated in Table 3-13 below.

Table 3-13 CVC lift joints' direct tensile strength

Dam zone	Designation	Split tensile strength of parent material (cylinders)	Lift joint direct tensile strength
		$f_{st}' = 0.12f_c'$ (MPa)	$f_t' = 0.08f_c'$ (MPa)
1	CVC	2.16	1.44

(d) Lift Joints' Shear Strength Parameters

The shear strength parameters (c' and ϕ') shown in table below were estimated for the different classes of concrete based on the one-year unconfined compressive strength, f_c' , of concrete sample determined from laboratory tests (EEPCo, 2011).

Table 3-14 CVC horizontal lift joints shear strength parameters

CVC zone	Characteristic compressive strength (365 days)	Internal friction angle	Cohesion
	f_c' (MPa)	ϕ' (Deg.)	c' (MPa)
CVC1	>18	45	1.2

[Source: EEPCo, 2011]

3.4.5.2 Thermal Properties

The thermal properties of the CVC region of the RCC dam were determined based on laboratory testing by the Designer/Engineer of the project (Studio Pietrangeli's study, 2016), as presented in Table 3-15 below, and these values were utilized in this study.

Table 3-15 CVC thermal properties

Parameter	Designation	Unit	Value
Specific heat (C_h)	C_h	kJ/kg-°C	0.820
Thermal conductivity (k)	k	W/m-°C	0.890
Thermal diffusivity (h^2)	h^2	m ² /hr	0.0016
Coefficient of thermal expansion (α)	α	(x10 ⁻⁶) 1/°C	8.10
Adiabatic temperature rise (ΔT_{ad}) at the age of 365 days	ΔT_{ad}	°C	27.4

[Source: Studio Pietrangeli, 2016]

3.4.5.3 Dynamic Properties

The dynamic material properties for the idealized CVC dam were estimated based on the USACE (2007) guidelines as summarized in Table 3-16 below.

Table 3-16 CVC dynamic properties

Dynamic properties	Designation	Conversion factor*	Value
Dynamic modulus	E_d	1.15*	23000 MPa
Poisson's ratio	ν	0.70*	0.14
Compressive strength	f_c'	1.15*	20.70 MPa
Lift joints direct tensile strength	f_t'	1.50*	2.16 MPa

*Conversion factors applied to convert static to dynamic properties

For the foundation rockmass, the dynamic properties are assumed to be similar to the static properties.

3.4.5.4 Damping

Similar to the RCC dam, an effective structural damping ratio of 5% was considered in the dynamic analysis of the idealized CVC dam; and for the massless foundation, damping effect was neglected.

3.5 Design/Evaluation Earthquake Ground Motions

3.5.1 Selecting Design/Evaluation Earthquakes

The design earthquake ground motions used for the comparison of seismic performance of RCC and CVC dams are the Operating Basis Earthquake (OBE) and the Safety Evaluation Earthquake (SEE) ground motions, according to the ICOLD (2010) method.

The OBE and SEE ground motions were established based on the researcher's comparative analysis and reviews of the site-specific seismic hazard studies conducted by EEPCo (2007), Aman, A.(2020), and the regional seismic study by Ayele, A. (2017), as explained in section 3.3.4 (a), (b) and (c). Accordingly, in accordance with the recommendations stated in section 3.3.4 (d) for this particular study, the SEE and OBE ground motions estimated by Aman, A.(2020), presented in Table 3-17 below, were chosen and applied for developing the ground motion parameters and acceleration-time histories for dynamic analysis.

Table 3-17 Design earthquake ground motions

Design Earthquake	Average Return Period (T_R)	Probability of Exceedance (P_E)	Service life (T_L)	Damping ratio	Method of analysis	Estimated magnitude of PGA	Ground condition
SEE	10000yrs	-	-	5%	PSHA	0.34g	Average rock site Vs30 = 760 m/s
OBE	145yrs	50%	100	5%	PSHA	0.10g	

[Source: Aman, A., 2020]

3.5.2 Developing Site-Specific Response Spectra

The site-specific response spectra for Gibe III dam site was developed based on the research findings, uniform hazard spectra and recommendations made by Aman, A.(2020) using PSHA, as discussed in section 3.3.4(d). This study was made considering an effective structural damping ratio of 5%, shear wave velocity of 760m/s, and a dam site of generic rock outcrops.

Accordingly, the target response spectra for both horizontal acceleration and vertical acceleration, illustrated in Figure 3-9 (for OBE, 145 years) and Figure 3-10 (for SEE, 10000 years), were produced in accordance with the guidelines provided in the Ethiopian Code ES EN 1998: 2015 [Design of Structures for Earthquake Resistance, Part 1: General Rules].

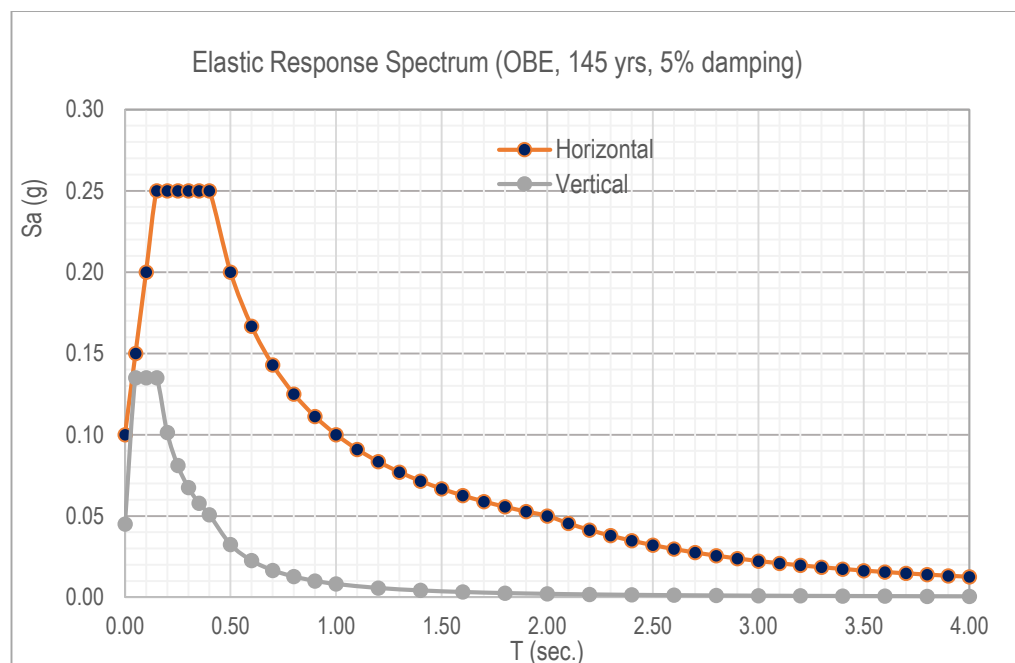


Figure 3-9 Target response spectra: OBE horizontal and vertical

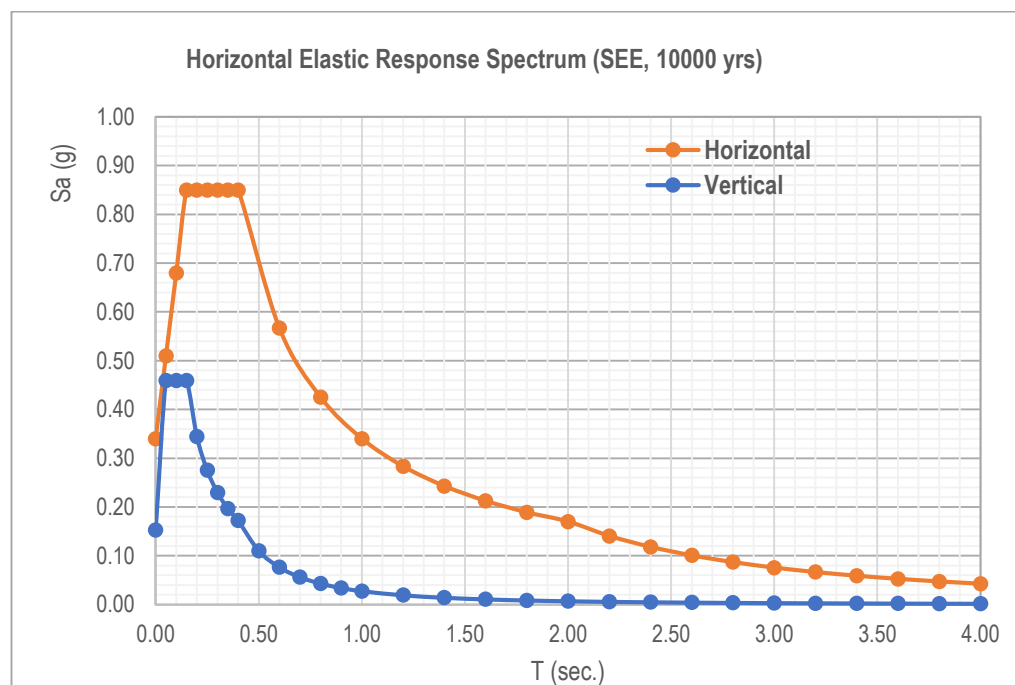


Figure 3-10 Target response spectra: SEE horizontal and vertical

Parameters describing the horizontal elastic response spectrum: both OBE and SEE

Ground Type	S	T_B	T_C	T_D
A (rockbed)	1	0.15	0.4	2

PGA for OBE at 145 Yrs return period = 0.10g

PGA for SEE at 10,000 Yrs return period = 0.34g

Symbols used:

S_a : the elastic response spectra (acceleration)

T : the vibration period of a linear SDOF system

a_g : the design ground acceleration, i.e, PGA

T_B : the lower limit of the period of constant spectral acceleration branch (sec.)

T_C : the upper limit of the period of constant spectral acceleration branch (sec.)

T_D : the value defining the beginning of the constant displacement response range of spectrum

3.5.3 Developing Acceleration-Time Histories for Dynamic Analysis

The acceleration time-histories of ground motion for dynamic analysis of both types of dams were developed according to the procedures outlined in USACE (2003) guidance, EM 1110-2-6051.

(a) Selection of Representative Records of Acceleration-Time Histories

The following criteria were established for initially selecting a set of 20 records of candidate acceleration-time histories, from the online PEER ground motion database tool (<https://ngawest2.berkeley.edu>), that are fairly representative of the site ground motions and also in consideration of the types of analyses planned for the dam.

Table 3-18 Summary of PEER ground motion database initial search criteria

Selection parameter	Search criteria	
	OBE case	SEE case
Target spectrum (horizontal)	Refer to Figure 3-9	Refer to Figure 3-10
Fault Type	Strike slip + normal	Strike slip + normal
Magnitude	5 - 8	5 - 8
R _{JB} (km)	10 – 200	5 - 300
R _{rup} (km)	10 – 200	5 - 300
D5-95(sec)	-	-
Vs30 (m/s)	700 - 1000	700 - 1000
Pulse	No pulse like records	No pulse like records
Initial scale factor	0.5 – 2.5	No pulse like records
Damping ratio	5%	5%

Accordingly, for use in the linear dynamic analysis, one set of four records of ground motion database for OBE loading case, and another set of four records for SEE loading case, were selected from PEER database, as summarized in Table 3-19 and Table 3-20 below.

Table 3-19 Summary of a set of initially selected unscaled ground motion database: PEER [OBE case]

Result ID	Record Sequence Number	Year	Earthquake Name	Station Name	Magnitude	Mechanism	Rjb (km)	Rrup (km)	Vs30 (m/sec)
1	RSN 454	1984	"Morgan Hill"	"Gilroy - Gavilan Coll." California	6.19	Strike-slip	14.83	14.84	729.65
2	RSN 476	1984	"Morgan Hill"	"UCSC Lick Observatory" California	6.19	Strike-slip	45.47	45.47	713.59
3	RSN 4312	1984	"Umbria-03_ Italy"	"Gubbio" Italy	5.6	Normal	14.67	15.72	922
4	RSN 4554	2009	"L'Aquila (aftershock 2)"	"L'Aquila - Parking" Italy	5.4	Normal	13.99	19.97	717

Table 3-20 Summary of a set of initially selected (unscaled) ground motion database: PEER [SEE case]

Ser. No.	Record Sequence Number	Year	Earthquake Name	Station Name	Magnitude	Mechanism	Rjb (km)	Rrup (km)	Vs30 (m/sec)
1	RSN 1161	1999	"Kocaeli_ Turkey"	"Gebze"	7.51	strike slip	7.57	10.92	792
2	RSN 1633	1990	"Manjil_ Iran"	"Abbar"	7.37	strike slip	12.55	12.55	723.95
3	RSN 1787	1999	"Hector Mine"	"Hector"	7.13	strike slip	10.35	11.66	726
4	RSN 3925	2000	"Tottori_ Japan"	"OKYH07"	6.61	strike slip	15.23	15.23	940.2

(b) Spectral Matching of the Recorded Acceleration-Time Histories

Modification of the recorded acceleration time-histories was done using Spectral Matching approach with the help of a computer software called SeismoMatch 2022 to develop the final sets of time-histories to be used in the response dynamic analysis of both types of dams. The spectrum matching involves modifying the frequency content of a given time-history so that its response spectrum is a close match to the design response spectrum.

(c) SeismoMatch Software Outputs

SeismoMatch is an earthquake response matching program that applies adding or subtracting elementary wavelets to / from the initial earthquake accelerograms so as to fit a particular target response spectrum using either the wavelets algorithm described by Al Atik and Abrahamson [2010] or the algorithm proposed by Abrahamson [1992] and Hancock et al.(2006). In this approach, the selected accelerograms were first loaded and then the target response spectrum was set. Thenafter, the spectral matching algorithm developed by Al Atik and Abrahamson [2010] was selected; the spectral matching period range 0.05 to 4.0 sec. was established; and also a tolerance limit of 25% allowed. Finally, the spectral matching was executed; and convergence with the design spectrum was confirmed by checking that the spectral values of the modified time-history are within specified deviations from the design values at every period being matched.

Selected SeismoMatch outputs showing original and matched accelerograms, together with target spectrum, are illustrated in Figure 3-11 (a) to (d) below.

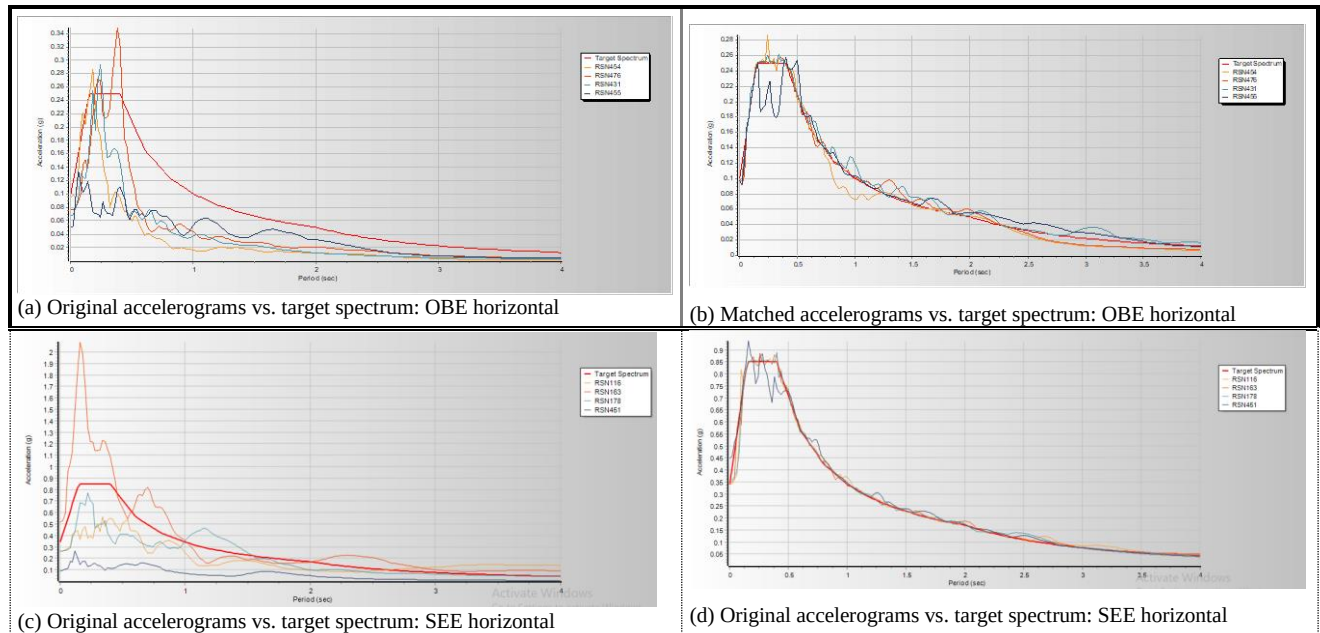


Figure 3-11 Original accelerograms, matched accelerograms and target spectra after processing using SeismoMatch software

(d) Recommended Matched Acceleration-Time Histories for Use in Time-History Analysis

Of all the records of accelerograms selected from PEER data ground motion database, those presented in Table 3-19 for OBE loading case and those in Table 3-20 for SEE loading case, the two accelerograms, namely RSN 454_MORGAN_GIL337.AT2] and RSN 1633_MANJIL_ABBAR--L.AT2, were found to be best fitting to the target response spectra produced for OBE and SEE loading cases, respectively. The original and matched accelerograms of these two records were then utilized in this study, and their matched accelerograms produced using SeismoMatch software are illustrated in Figure 3-12 (a) to (d) below.

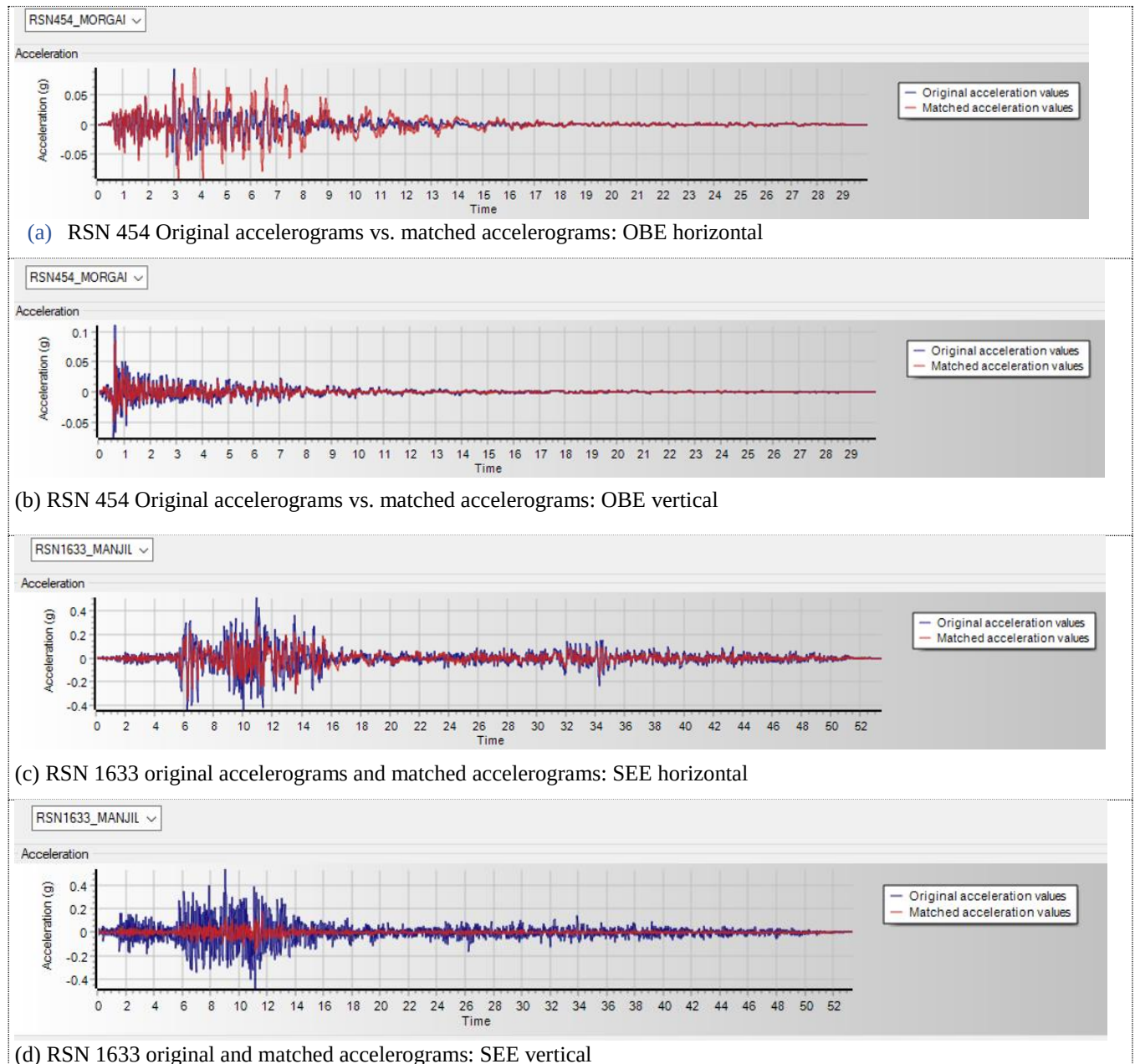


Figure 3-12 RSN 454 and RSN 1633: original accelerograms vs. matched accelerograms

The time series database for the originally recorded accelerograms for horizontal excitations used in this study (RSN 454_MORGAN_GIL337.AT2, for OBE loading case] and RSN 1633_MANJIL_ABBAR--L.AT2, for SEE loading case] re presented in Appendix-I.

(e) Further modification of the Acceleration-Time Histories for Site Response Effects

No further modification of the time-histories was considered due to insignificant site response effects, spatial variations of ground motions and wave transmission mechanisms.

3.5.4 Proposed Earthquake Ground Motion Directions

In this study, only horizontal component of the design/evaluation earthquake ground motion was considered in strength evaluation using SEE as well as in serviceability evaluation using OBE.

Accordingly, upstream-downstream and downstream-upstream directions were considered in the case of an empty reservoir and full reservoir conditions, as shown in **Table 3-21** below

Table 3-21 Summary of horizontal earthquake ground motion directions

Load case.	Earthquake loading	Loading condition	Ground motion direction
1	OBE	Reservoir empty case	Downstream-upstream
2	OBE	Reservoir full (operating level)	Upstream-downstream
3	SEE	Reservoir full (operating level)	Upstream-downstream

3.6 Selection of Earthquake Response Analysis Procedures

A linear-elastic time-history analysis methodology based on 2-D finite element procedures, as discussed in Section 2.9, was adopted for the earthquake response analysis of both types of dams. The finite element models for the concrete dams, foundation rock, and the impounded water as well as loading combinations were developed according to the prescribed guidelines and criteria provided in USACE 2003, EM 1110-2-6051.

- (a) The geotechnical properties of the foundation rockmass were established based on the geotechnical investigations, findings and recommendations (EEPCo, 2008), and the concrete elastic properties were extracted from the full-scale trials and laboratory and/or in-situ tests (EEPCo, 2011), both of which were produced for the design stage of the project. The dynamic material properties were estimated to reflect the appropriate increases due to rapid rate of seismic loading, according to the recommended guidelines in USACE 2007, EM 1110-2-6053.
- (b) The ground motion acceleration time-history inputs for the dynamic analysis were developed or selected based on the site-specific response spectra (uniform hazard spectra) produced by Aman, A.(2020) for estimating the SEE, with 10000-year return period, and OBE, with 145 return period, at the existing dam site of the Gibe III hydroelectric plant.
- (c) An incompressible reservoir (fluid) was employed. The interaction effects of the impounded water and dam were included in dynamic analyses using the finite element formulation of the hydrodynamic added mass concepts according to the Generalized Westergaard's approach (Kuo, 1982), discussed in Chapter 2.
- (d) The foundation was assumed flexible but massless, and the interaction between the dam and foundation was represented by bonded or frictional interface where the Mohr-Coulomb shear parameters (cohesion and angle of internal friction) were utilized.
- (e) Structural damping effects (5% of the critical damping) are included in the dam structure only using a viscous damping coefficient proportional in scale to the severity of dam shaking during the earthquake.

- (f) The performance of both types of dams was evaluated based on the contours showing the extreme and instantaneous stress distribution over the entire cross-sectional area of each dam type at critical instants of time, as well as stress time-histories indicating magnitude and number of excursions of peak stresses beyond maximum allowable values and their relationship to surrounding stresses.

3.6.1 Loads and Load Combinations

3.6.1.1 Static Loads

The following static loads were considered in the study. The types and computations of magnitudes are presented in Table 3-22 through to Table 3-27 below, and the loads acting on the dam are schemetically presented in Figure 3-13.

- (a) **Dead loads (GW).** The dead load of the dam structure is calculated for a unit length (1m) of the dam as required for 2-D structural analysis. The weight per linear meter is calculated as the product of the gross cross-sectional area of the dam and the unit weight of the concrete.

Table 3-22 Gross weight of the dam structure

Gross cross-sectional area of dam structure	$A_{st} = 22268.5 \text{ m}^2$
Unit weight of concrete	$\gamma_c = 23.5 \text{ KN/m}^3$
Gross-weight of dam per unit length (1m slice)	$GW = \gamma_w * A_{st} = 523309.5 \text{ KN/m}$

- (b) **Reservoir Loads.** Reservoir loads are calculated as linearly varying hydrostatic pressures on the upstream dam face in both horizontal and vertical directions

Table 3-23 Hydrostatic pressures in horizontal direction

Unit weight of water	$\gamma_w = 10 \text{ KN/m}^3$
Maximum depth of water above reservoir bottom	$H = 222 \text{ m}$
Hydrostatic pressure at water surface level	$p_{wh1} = 0 \text{ KN/m}^2$
Silt pressure at reservoir bottom	$P_{wh2} = \gamma_w * H = 2220 \text{ KN/m}^2$

Table 3-24 Hydrostatic pressures in vertical direction

Upstream dam face nodes		Water depth below water surface	Vertical hydrostatic pressure, p_{wv}
$x_i \text{ (m)}$	$y_i \text{ (m)}$	$h_i \text{ (m)}$	$\text{(KN/m}^2\text{)}$
0.00	0	222	2220.00
14.32	40.91	181.09	1810.90
29.09	99.98	122.02	1220.23
39.88	208	14.00	140.00
39.88	222	0.00	0.00

- (c) **Silt pressures:** Silt pressures in horizontal direction are calculated as linearly varying pressure loads, with zero magnitude at top and maximum value at reservoir bottom, on the upstream face of the dam, as below

Table 3-25 Silt pressures in horizontal direction

Saturated unit weight of silt (sediments)	$\gamma_{sat} =$	17 KN/m ³
Unit weight of water	$\gamma_w =$	10 KN/m ³
Submerged unit weight of silt	$\gamma_{sub} = \gamma_{sat} - \gamma_w =$	7 KN/m ³
Estimated elevation of silt deposits after 50years (EEPCo,2013)	Elev=	689 masl
Base (foundation) of dam	Elev=	670 masl
Estimated depth of silt (sediment) in reservoir	Hs=	689-670=19 m
Angle of internal friction (EEPCo, 2008)	$\Phi =$	25 Deg.
Lateral earth pressure coefficient (at-rest condition)	$K_o = 1 - \sin(\Phi) =$	0.577
Silt pressure at top level	ps=	0 KN/m ²
Silt pressure at reservoir bottom	$ps = \gamma_{sub} * H_s * K_o =$	76.79 KN/m ²

- (d) **Uplift Pressures.** Uplift pressures computed for reservoir full (operating level) condition, in accordance with the USACE 1995 guidelines, for the case of drainage gallery close to the upstream dam face (i.e. $H_4 < H_2$), are presented in Table 3-26 below.

Table 3-26 Uplift pressure distribution according to USACE (1995) guidelines

Uplift pressure distribution condition	Bottom level of drainage gallery at lower elevation than than the normal tailwater level (i.e $H_4 < H_2$)	
Total depth of headwater/ reservoir	$H_1 = H =$	222 m
Depth of tailwater	$H_2 =$	7.00 m
Base width of dam structure	$L =$	193 m
Height of drainage gallery measured from base of dam	$H_4 =$	2.00 m
Location of drainage gallery from upstream face bottom, X=	$X =$	20.00 m
Drain effectiveness: $K = 0.25 - 0.50$, assumed $K =$	Assumed $K =$	0.40
Estimated uplift pressure head at drainage gallery location (i.e. at x-distance from upstream end)	$H_3 = K (H_1 - H_2) * (L - X) / L + H_2 =$	84.00 m
Unit weight of water	$\gamma_w =$	10 KN/m ³
Computed uplift pressure at upstream face /heel point/	$u_1 = \gamma_w * H_1 =$	2220 KN/m ²
Computed uplift pressure at drainage gallery location	$u_x = \gamma_w * H_3 =$	840 KN/m ²
Computed uplift pressure at downstream end /toe/	$u_2 = \gamma_w * H_3 =$	70 KN/m ²

- (e) **Tailwater pressures:** Tailwater pressures are calculated at normal pool (operating) reservoir condition as linearly varying load as presented in Table 3-27 below

Table 3-27 Tailwater pressure at normal pool (operating) reservoir condition

Unit weight of water	$\gamma_w =$	10 KN/m ³
Estimated depth of normal tailwater	$h_{tw} =$	7 m
Tail water pressure at water surface level	$p_{tw} =$	0 KN/m ²
Tail water pressure at downstream bed level	$p_{tw} = \gamma_w * h_{tw} =$	70 KN/m ²

3.6.1.2 Secondary Loads

(a) Wind Pressures. The wind load acting on the exposed surface (unit or 1m width) of both types of dams was estimated using the guidelines and equations provided in the Compulsory Ethiopian Standard, CES 145 (2015). The wind pressure was estimated to act on the upstream face of the dam above the normal pool (operating) water surface assuming that the effects of wind pressure variation with height of structure, terrain roughness, terrain orography, and wind turbulences are neglected. The reference basic wind speed at Gibe III dam site was taken to be 35m/s according to the wind speed distribution map produced by Melesse, G., Melesse, H., Devadass T. (2021).

Table 3-28 Wind pressure acting on the dam at empty and normal pool (operating) reservoir condition

Parameter	Designation	Result	Remark
Density of air	$\rho_a =$	1.25 Kg/m ³	CES 145 (2015)
Estimated height of exposed upstream face			
- Empty reservoir case	$h_{emp} = 896-670 =$	226 m	EEPCo (2013)
- Normal pool (operating) reservoir case	$h_{norm} = 896-892 =$	4 m	
Reference basic wind velocity	$v_b =$	35 m/s	Melesse, G., Melesse, H., & Devadass T. (2021)
Mean wind velocity	$v_m \approx v_b =$	35 m/s	CES 145 (2015), assuming terrain roughness, orographic effects, turbulence effects neglected
Peak wind velocity pressure	$q_p = (1/2) \cdot \rho_a \cdot v_m^2 =$	766 Pa	CES 145 (2015)
	$=$	0.766 KN/m ²	
Total maximum wind pressure (force) per meter length of dam			
- Empty reservoir case	$P_{empty} = q_b \cdot h_{empty} =$	173.0 KN/m	
- Normal pool (operating) reservoir case	$P_{normal} = q_b \cdot h_{normal} =$	3.10 KN/m	

As can be seen in the above table, the peak velocity pressure acting on the dam was estimated to be 0.766 KN/m², which is too small as compared to the other static loads calculated above in section 3.6.1.1 (a) to (d). The total pressure (force) calculated per meter length of dam was also very small (i.e 173 KN/m for reservoir empty case, and only 3.1 KN/m for normal pool condition). Hence, the wind load effects were neglected in this study.

(b) Wave Action. The total wave force, P_{wv} (in kN), was estimated according to the guidelines and the empirical methods described in section 2.5.7(b), as per the formula provided in IS:6512-1984 (reprinted 1998) and Novak et al. (2007).

Table 3-29 Estimated wave pressure acting on the dam at normal pool (operating) reservoir condition

Parameter	Designation	Result	Reference
Unit weight of water	$\gamma_w =$	10 KN/m ³	
Estimated maximum wind velocity	$U = v_m = 35 \text{ m/s} =$	126 Km/hr	Chapter 2, Section 2.5.6 (a)(iv), page 59
Fetch (estimated based on measurement of reservoir area on Google Earth Map)	$F =$	12 Km	Google Earth Map
Estimated height of wave at u/s dam face ($F < 32 \text{ km}$)	$h_w = 0.032 \cdot (U \cdot F)^{0.5} + 0.763 - 0.271(F)^{0.25} =$	1.5 m	Chapter 2, Section 2.5.6 (b)(ii) and (iii), page 59-60
Total height of wave pressure envelope	$H_{wv} = 5/3 \cdot h_w =$	2.5 m	
Maximum wave pressure intensity	$p_{wmax} = 2.4 \gamma_w \cdot h_w =$	36.1 KN/m ²	
Total horizontal force due to wave action per meter length of dam length	$P_{wmax} = 2 \gamma_w \cdot h_w^2 =$	45.2 KN/m	

From the above calculation (Table 3-29), we can see that the total horizontal force due to wave action per meter length of dam at normal pool reservoir condition (**45.2 KN/m**) is comparatively much higher than the total wind pressure (force) per meter length of dam estimated (i.e. **3.1 KN/m**) calculated in Table 3-28 for the same reservoir condition. The total force due to wave action is still small as compared to the other static loads calculated in section 3.6.1.1. The wave pressure may be considered in places with higher basic wind velocities together with the other static loads in the dynamic analysis. However, in this study, the wave action effects were neglected in this study.

(c) Thermal Loads. Thermal effects were not included in the seismic evaluation of both types of dams, as per the recommendations provided in USACE 1995, EM 1110-2-2200.

(d) Ice Loads: Ice loads were neglected in the seismic evaluation in this study, as per the recommendations provided in USACE 1995, EM 1110-2-2200.

3.6.1.3 Earthquake Loads

The design/evaluation earthquake ground motions, including the proposed OBE and SEE ground motions, site-specific response spectra, the scaled/matched/ acceleration-time histories, and recommended earthquake ground motion directions for use in the dynamic analysis of both types of dams are discussed in section 3.5.

3.6.1.4 Hydrodynamic Loads

The hydrodynamic interaction effects between the dam and reservoir were investigated using the simplified added mass concepts according to the Generalized Westergaard Approach (Kuo, 1982), as discussed in section 2.5.9.3. The reservoir on upstream side, assumed incompressible, was added to the dam upstream face as a virtual lumped mass, as illustrated in Figure 3-13(a) below. Accordingly, the magnitudes of the added masses at different upstream dam face points - designated 'm1' to 'm14', for a unit slice (1m) length of reservoir, were calculated as presented in Table 3-30 below.

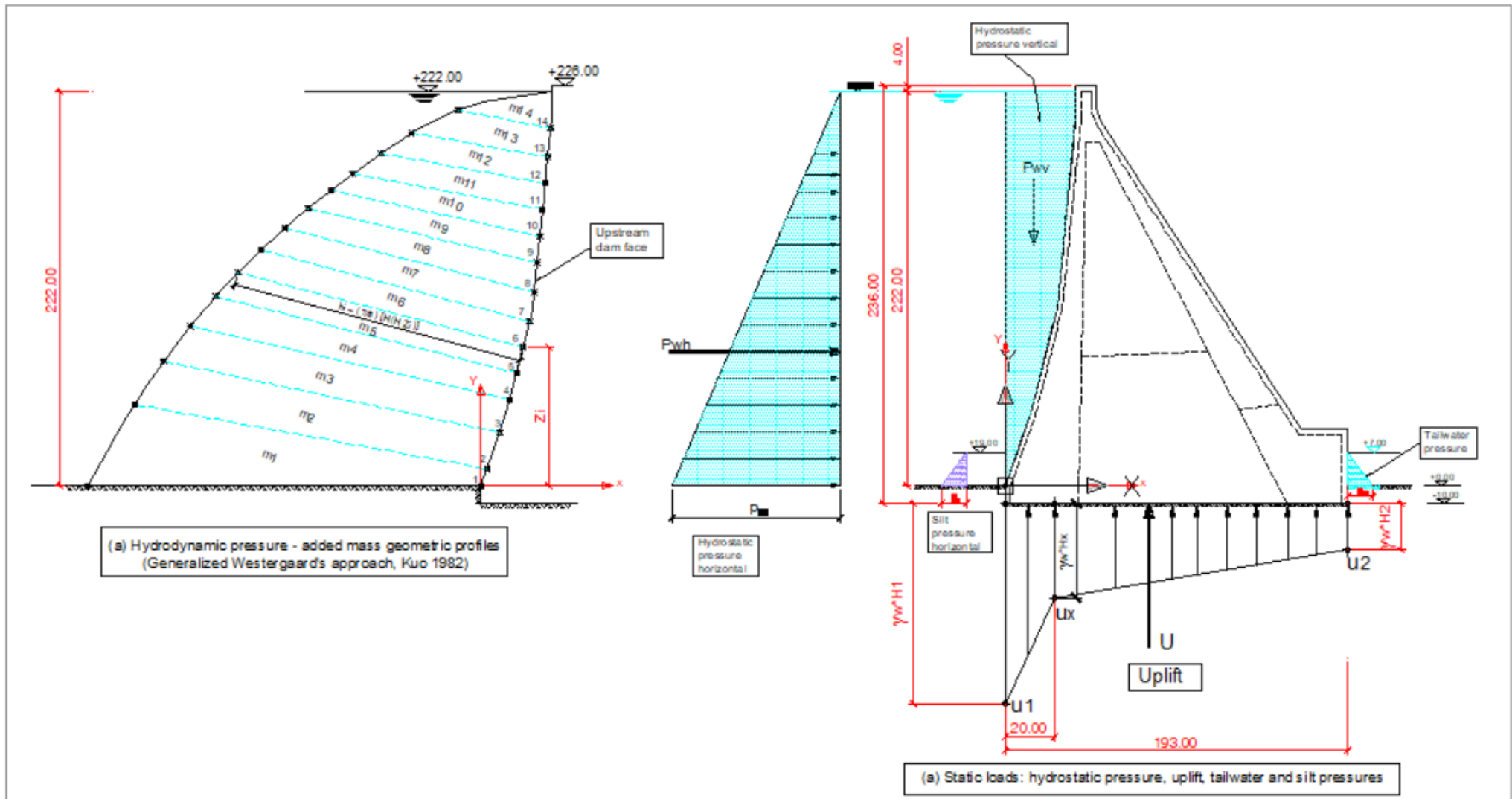


Figure 3-13 Schematic diagram of static loads acting on the dam and hydrodynamic added mass model

Table 3-30 Estimation of hydrodynamic added masses (Generalized Westergaard model, 1982)

Node, ni	Added mass C.G.		Height of node 'i' above reservoir bottom, Zi	Depth below water surface, Hi =H-Zi	Width of unit slice of added mass, bi	Added mass slice area, Ai	Added mass at node 'i', $m_i = A_i \cdot \rho_w$
	X- Coordinate	Y- Coordinate					
	(m)	(m)					
1	0.00	0.00	0.00	222.00	194.25	6098.931	6098931.40
2	6.65	19.00	19.00	203.00	185.75	4811.132	4811132.10
3	14.32	40.91	40.91	181.09	175.44	3984.034	3984034.00
4	18.17	56.32	56.32	165.68	167.81	3160.325	3160325.10
5	21.81	70.87	70.87	151.13	160.27	2655.154	2655154.30
6	25.45	85.42	85.42	136.58	152.36	2467.153	2467152.70
7	29.09	99.98	99.98	122.02	144.01	2427.501	2427500.60
8	30.93	118.40	118.40	103.60	132.70	2118.732	2118732.10
9	32.42	133.33	133.33	88.67	122.77	1746.507	1746507.10
10	33.91	148.25	148.25	73.75	111.96	1525.485	1525484.90
11	35.41	163.18	163.18	58.82	99.99	1473.601	1473600.70
12	36.90	178.11	178.11	43.89	86.37	1252.982	1252981.50
13	38.39	193.03	193.03	28.97	70.17	1113.447	1113447.20
14	39.88	207.96	207.96	14.04	48.86	598.643	598642.70
	Total added mass of water (Kg) =						35,433,626.40
	Total added mass of water (ton.) =						35,433.63

3.6.1.5 Load Combinations for Earthquake Time-History Analysis

About three earthquake loading conditions, listed below, which USACE (1995) suggests as being sufficient combinations for almost all circumstances, were proposed for the 2-D linear-elastic time-history analysis of both types of dams, with only horizontal excitation considered, as presented in Table 3-31 below.

- 1) **Load Combination-1** [Extreme Loading Condition]: Dam construction completed, empty reservoir case, subject to OBE ground motions in upstream direction.
- 2) **Load Combination-2** [Unusual Loading Condition]: Dam under normal operation, full reservoir condition, subject to OBE ground motions in downstream direction.
- 3) **Load Combination-3** [Extreme Loading Condition]: Dam under normal operation, full reservoir condition, dam subjected to SEE ground motions in downstream direction.

Table 3-31 Earthquake loading conditions for time-history analysis (USACE 1995, EM 1110-2-2200).

Load class	Load source	Qualification	Earthquake Loading Combinations		
			Load Comb.1: [Extreme Loading Condition]	Load Comb.2: [Unusual Loading Condition]	Load Comb.3: [Unusual Loading Condition]
Primary loads	Head water	Empty Reservoir	+1		
		Maximum Water / Flood Level (MWL)			
		Full (normal) reservoir level (NWL)		+1	+1
	Tail water	Maximum tail water level			
		Normal tailwater level			
		Minimum tail water level		+1	+1
	Dead-weight	Self-weight vertical	+1	+1	+1
	Uplift	Drains operating		+1	+1
		Drains inoperative			
Secondary loads	Silt load	Silt load horizontal		+1	+1
		Silt load vertical		+1	+1
Exceptional loads	OBE-horizontal	Upstream-downstream direction		+1	
	SEE-horizontal	Upstream-downstream direction			+1

3.6.2 Finite Element Models and Analysis Using Ansys Workbench 2023

3.6.2.1 Structural Models

The linear-elastic time-history analysis of both types of dams was carried out using a 2-D plane-strain standard finite element model of the dam-reservoir-foundation system, according to the guidelines provided in USACE (2003), EM 1110-2-6051.

3.6.2.2 Modeling of Main Dam Components and Boundary Conditions

The entire dam-reservoir-foundation system is idealized as a composite system of 2-D plane-strain elements with constituent materials considered to be linearly elastic, isotropic, and homogenous.

(a) Foundation: Massless foundation model is included directly in the numerical model, with inertia and damping effects neglected. The length of the foundation rockmass was taken one dam height (236m) in the downstream direction, 1.5 times dam height (354m) in the upstream direction, and one dam height (236m) in the downward direction. The foundation was modelled to have fixed

(rigid) base at bottom nodal points, and roller supports at side nodal points to allow horizontal displacement in the x-direction

Foundation base nodes: $u_x = 0; u_y = 0$

Foundation side nodes: $u_x = \text{free}, u_y = 0$

(b) Dam: The dam is idealized as a 2-D plane-strain solid, isotropic, and homogeneous, linearly elastic body using quadrilateral (4-noded) finite elements.

(c) Reservoir: The inertia forces of the impounded water were represented by the hydrodynamic added mass values determined in accordance with the Generalized Wetgergaard's added mass model as illustrated in Figure 3-16 below. The hydrodynamic added masses were modelled in Ansys program as "point masses" being coupled/lumped to the dam upstream face and have the same DOFs as the main dam body.

3.6.2.3 Element Discretization (Meshing)

Both types of concrete gravity dams (RCC and CVC) were defined with solid 2-D plane-strain finite elements. For this case study, quadrilateral dominant (four-nodes) and triangular (3-noded) finite elements were defined to provide smooth or gradual transition of element sizes from small elements on the boundaries to large elements on the interior zones of the dam and the foundation model. Accordingly, the foundation was defined with coarser meshes (element size 15m) and the interior regions were provided with relatively finer meshes (5m to 9m).

Mesh quality check was performed with the ANSYS program 2023 to increase the accuracy of the numerical analysis

- 1) The discretized finite elements and nodes were set to recommended sizes and to a higher number as follows:

RCC dam meshing: mesh sizes range from 5m to 9m for the regions RCC1 to RCC5, and 15m for the massless foundation model; total number nodes 11004; and total number of elements 3579.

CVC dam meshing: uniform mesh and local minimum size of 5m for the entire CVC region, 15m for massless foundation model; total number of nodes 10021; total number of elements 3037

- 2) The use of distorted elements (for example, highly skewed elements) was, as much as possible, avoided in the element discretization (meshing) process, and an attempt was made to check that an aspect ratio close to one in most of the regions of the dam-foundation system was achieved in the mesh formulation.

- 3) Finally, a mesh quality was checked through convergence test that was performed by considering the use of different element discretization (mesh sizing) and varying element types (triangular vs quadrilateral elements) to confirm that the differences between the two approaches is so small or they were found close to each other (error less than 10%).
- 4) The minimum mesh size set for linear analysis meets the USBR's (2006) recommended minimum element size of 5m.

The typical 2-D plane-strain finite-element model for both RCC and CVC dams are shown in Figure 3-14 and Figure 3-15 below, respectively. The hydrodynamic added masses, modeled as point masses and lumped onto the dam upstream face, are shown in Figure 3-16 below.

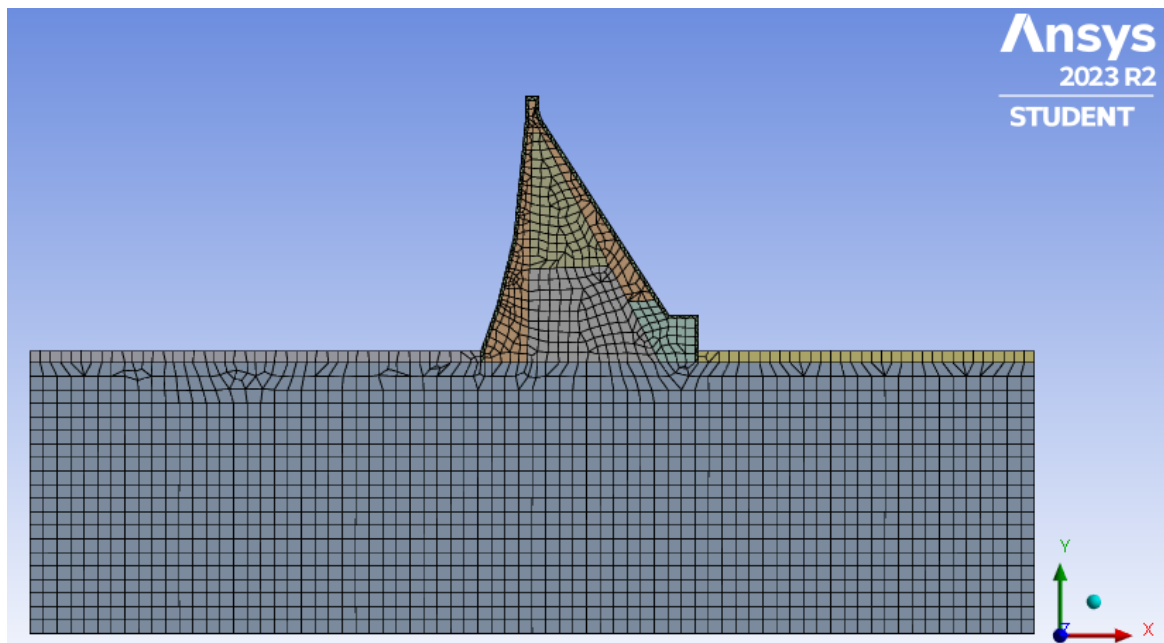


Figure 3-14 Standard finite element model of the existing RCC dam

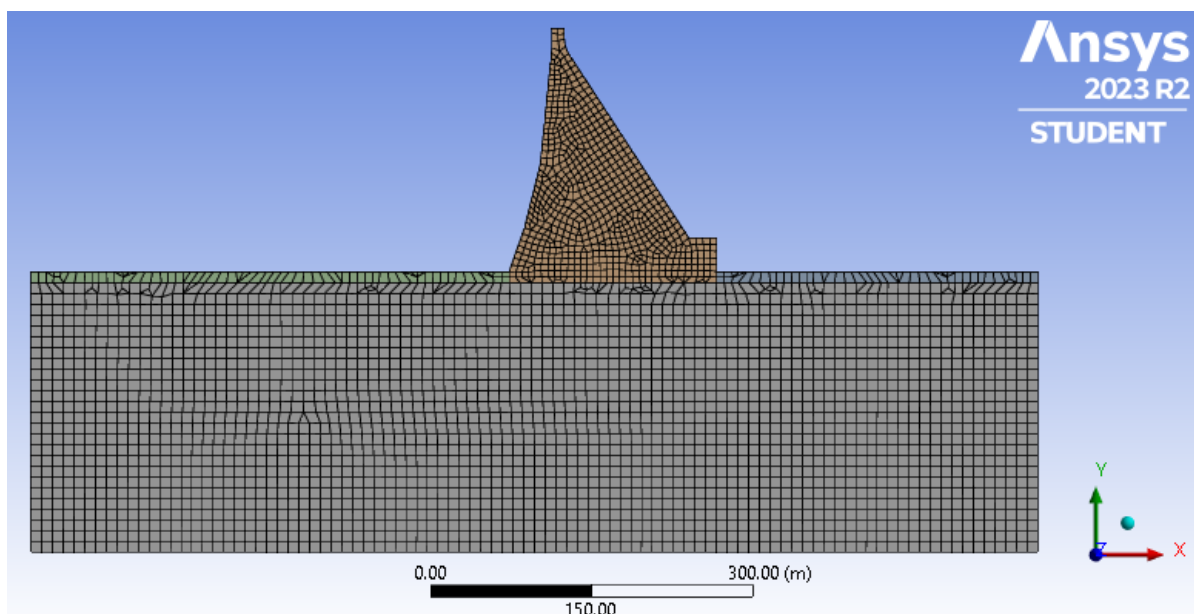


Figure 3-15 Standard finite element model of the idealized CVC dam.

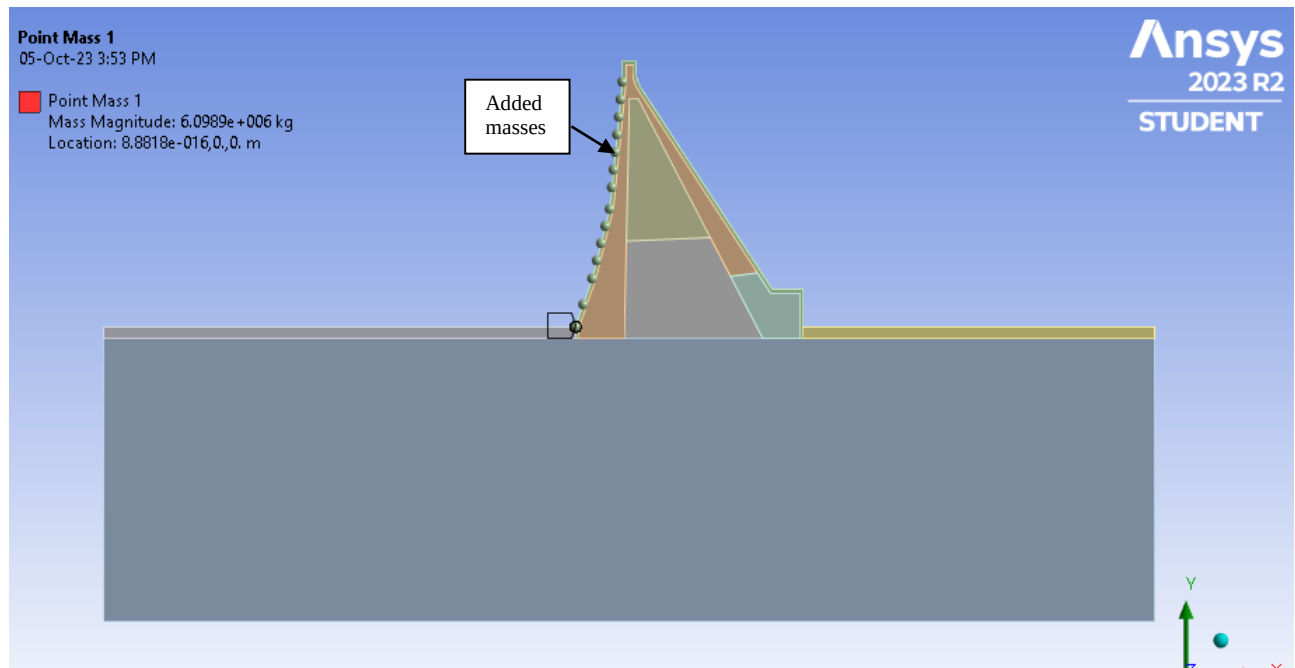


Figure 3-16 Hydrodynamic added masses lumped onto the nodes of the dam upstream face (typical)

3.6.2.4 Modeling Contacts

The behavior and interactions of contacts between different regions of the dam-reservoir-foundation system were defined using the following contact formulations in the ANSYS program 2023.

- **Bonded contact:** The different zones within the main dam body are connected to each other using the “bonded” type of contact.
- **Frictional contact:** The connection between the different zones of the main dam to the foundation are modelled as frictional interface.

3.6.2.5 Modeling Joints and Liftlines

(a) **Dam-foundation Joints:** three methods of contact modelling could possibly be utilized

- **Bonded contact:** The dam was assumed to be fully bonded to the rock foundation since concrete bonds well with rock surfaces that are properly cleaned (USBR, 2006).
- **Contact surface:** A more realistic modelling assumption (as proposed by Malm, 2016) is to define the contact between the dam and the rock foundation with contact surfaces, interactions, using a tie constraint.
- **Frictional interface:** The dam-foundation interfaces can be modelled as frictional contact type having a coefficient of friction of 1.0 assumed to exist between the concrete and sound rockmass (Malm, 2016a).

(b) Construction joints (Lift lines): The lift lines, critically weak and possible failure planes in the interior body of concrete dams, were modelled as load bearing stratum and bonded contacts having compressive strength equal to the parent concrete strength. However, the shear strength parameters determined through testing (EEPCo, 2011), and reduced direct tensile strengths, as presented in section 3.3.5 for the existing RCC dam and in section 3.4.5 for the idealized CVC dam, were used.

3.6.2.6 Modeling Uplift Pressures

The uplift pressures were computed as static loads and defined in the Ansys program as surface pressures. They were applied to opposite faces within the contact surfaces along the base of the dam, as per the guidelines provided in USBR (2006). The interface between the dam and foundation was modeled using contact surfaces (elements).

3.6.2.7 Damping

An effective structural damping ratio of 5% was considered for the dam-reservoir-foundation interactions, with the exception that, for the massless foundation, damping effect was neglected.

Alternately, as per USBR (2018) guidelines, for the dam structure, the use of Rayleigh damping with α and β assumed to be 0.95 and 0.002, respectively, that approximately corresponds to 5 percent of the critical damping, could be adopted in the full transient analysis using ANSYS program.

3.6.3 Direct Integration Method of Solution

The ANSYS Workbench 2023 R2 uses the implicit type of numerical approximation called Newmark time integration as a default method for solving the equations of motion at discrete time-points in the linear-elastic range.

However, in case the analysis control extends to nonlinear analysis, the Newton-Raphson option is the default method in the ANSYS program. Both methods are discussed in Chapter 2. The time steps between successive time-points, as obtained from the unscaled ground motions downloaded from PEER ground motion database, were adopted.

The incremental load application and integration time step used in the direct time step integration is 0.005 seconds, as commonly recommended value by USBR (2006).

3.6.4 Prediction of Earthquake Responses

For seismic performance evaluation of the designated RCC and CVC gravity dams, the following results from linear time-history analyses were produced, in accordance with the requirements set in USACE (2007) for seismic performance evaluation of existing dams.

- 1) **Displacement histories:** magnitudes and time-histories of nodal displacements at critical locations such as the crest and intermediate locations of slope changes.
- 2) **Maximum and minimum principal stresses:** Maximum and minimum principal stresses due to static plus seismic loads were determined and presented as contours and vector plots. The maximum and minimum stress plots represent the largest tensile (positive) and the largest compressive (negative) stresses that occur in the dam generally at different times during the earthquake excitation.
- 3) **Demand-capacity ratios:** The maximum principal stresses in item no.(2) above were displayed as plots of the demand-capacity ratios by dividing the maximum stresses by the lift joints tensile strength and minimum stresses by the compressive strength of the concrete.
- 4) **Time-histories of displacement and acceleration histories:** Time histories of crest horizontal displacement and accelerations in the x-axis were produced and comparison made between the two types of dams when subjected to the prescribed earthquake load combination cases.
- 5) **Time histories of critical principal stresses:** Time-histories of the most critical maximum principal stresses identified in item no.(2) above were displayed and evaluated. Such time-histories were examined to determine the total duration of stress excursions beyond demand-capacity ratios of 1.0 to 2.0 for comparison with the performance curve in Figure 2-14 presented in chapter 2. The magnitude of cumulative inelastic duration indicates whether the peak stresses are merely spikes with little or no damage potential or they are of longer duration capable of producing significant damage.

3.6.5 Guidelines for Comparative Evaluation of Seismic Performance

The results of linear time-history analyses of both types of dams were presented side by side, and comparisons and evaluations were made in accordance with the USACE (2007) guidelines, EM 1110-2-1053, described in Chapter 2.

Finally, conclusions were drawn so as to justify which type of dam has superior or better seismic performance over the other one, and recommendations made in consideration of the Ethiopian planning and current practices in development of similar large storage dam projects.

4 RESULTS AND DISCUSSIONS

4.1 Scope

This section briefly summarizes and describes the results of the time-history analyses of the two types of dam-reservoir-foundation systems (RCC and CVC types) when subjected to the recommended earthquake loading conditions under OBE and SEE loadings, described in section 3.6.1.5, which combine the initial static load effects with the dynamic effects of the critical directions of horizontal excitations.

The analyses began by considering the initial static loadings with the critical directions of horizontal excitation in order to determine the critical loading condition that produces maximum tensile stress effects in the section since damage to a concrete dam is mainly linked to the development of tensile stresses in critical regions. In summary, the following sections describe the results of the linear-elastic time-history analysis using ANSYS workbench 2023 when both types of dams were subjected to the prescribed load combinations constituting OBE and SEE loadings for the associated reservoir conditions.

4.2 Directional (Horizontal) Displacements

Table 4-1 below presents the comparative dynamic responses in terms of horizontal displacements of the two types of dams when subjected to the proposed earthquake loading combinations. The horizontal deformations in the form of contour bands of maximum horizontal displacement are depicted in Figure 4-1, Figure 4-2 and Figure 4-3 for the corresponding load combination cases. As can be seen in this table, the peak values of horizontal dynamic displacement at the crest of both RCC and CVC dams for Loading Condition-3 [SEE loading, full reservoir case] is slightly higher than the corresponding values for Loading Condition-2 [OBE loading, full reservoir case] but much higher than that for Loading Condition-1 [OBE loading, empty reservoir case]. It can also be learnt that the maximum horizontal displacements that resulted from the three earthquake load combination cases have more or less similar trends under the three load combination cases with both types of dams.

Table 4-1 Maximum crest horizontal displacements under OBE and SEE ground motions

Loading condition	Earthquake ground motion	Reservoir condition	Maximum displacement		Reference
			RCC dam	CVC	
Load Combination1	OBE/upstream direction	Empty	6.06 cm at 0.275 sec	5.60 cm at 0.26 sec.	Figure 4-1 (a to d)
Load Combination2	OBE/downstream direction	Full (normal pool)	28.97cm at 0.365 sec.	27.74 cm at 0.355 sec.	Figure 4-2 (a to d)
Load Combination3	SEE/downstream direction	Full (normal pool)	29.02cm at 0.365 sec.	27.78 cm at 0.35 sec.	Figure 4-3 (a to d)

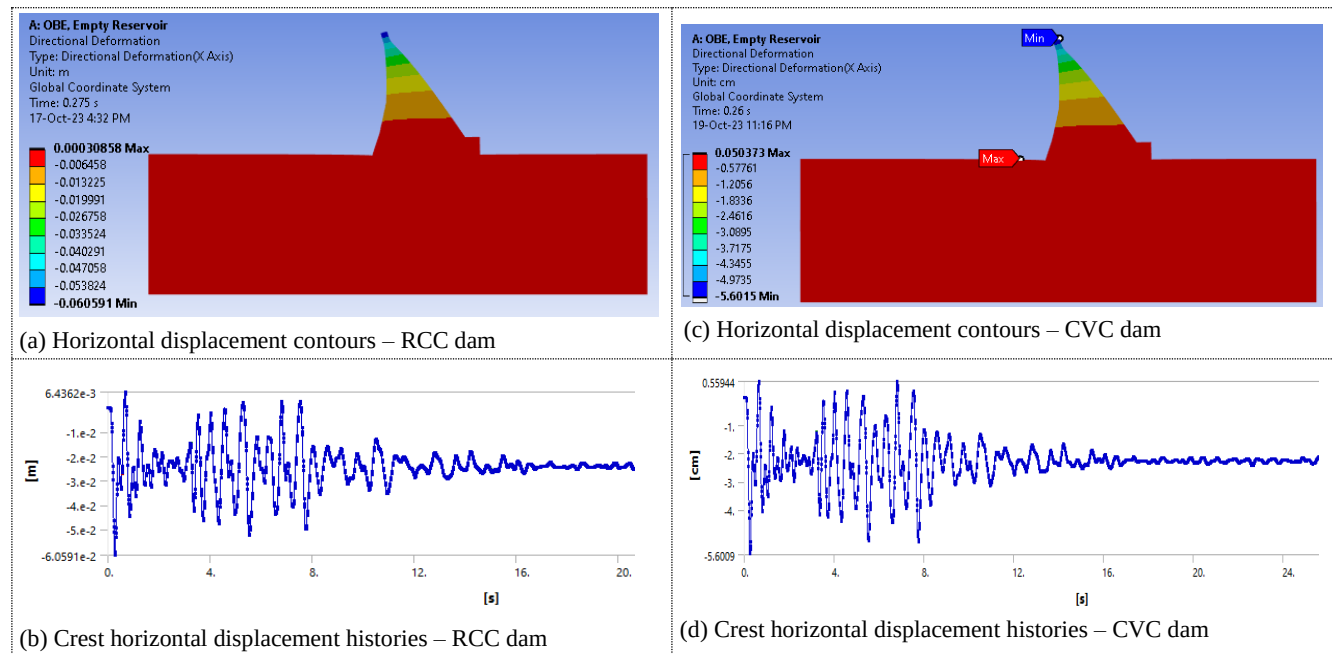


Figure 4-1 Horizontal displacements: OBE loading case, empty reservoir condition

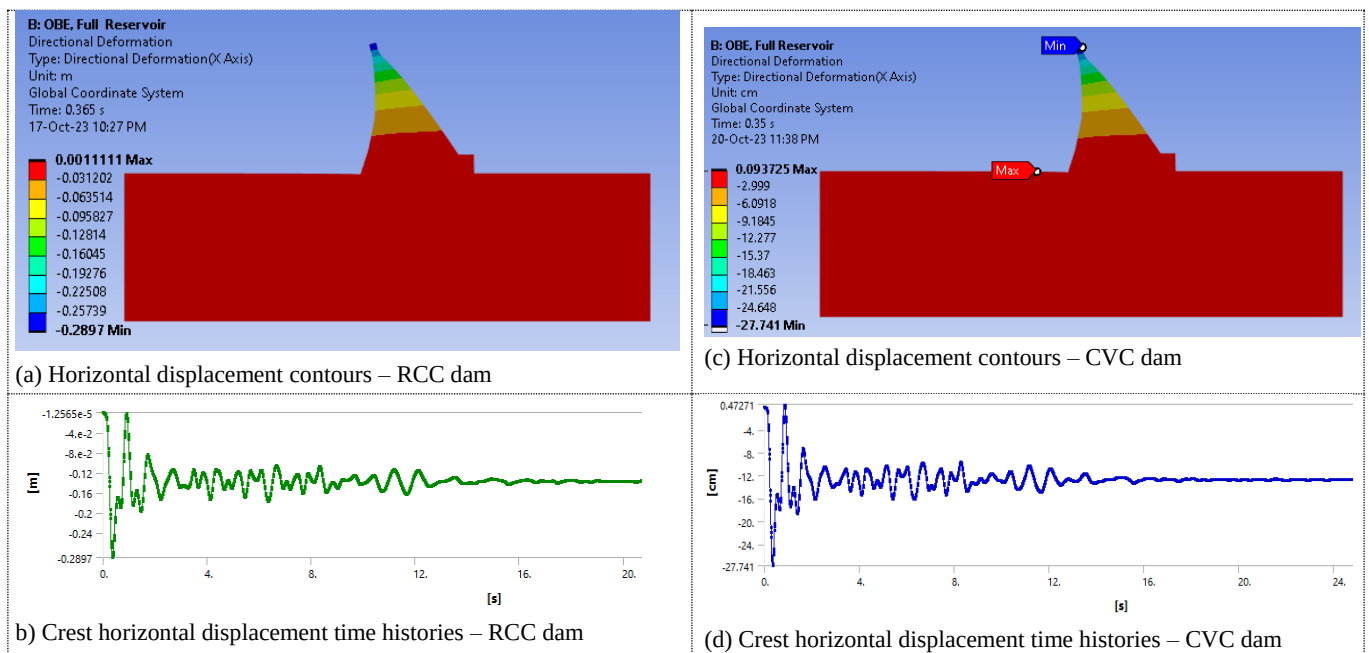


Figure 4-2 Horizontal displacements: OBE loading case, full reservoir condition

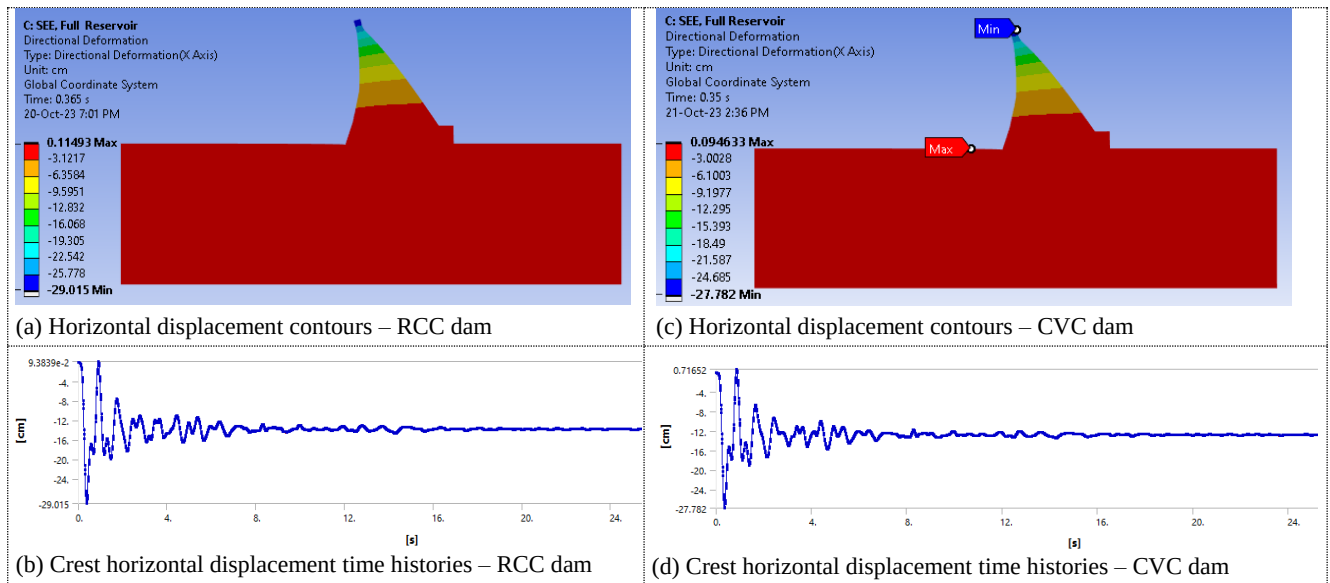


Figure 4-3 Horizontal displacements: SEE loading case, full reservoir condition

4.3 Directional (Horizontal) Accelerations

Table 4-2 below presents the ANSYS program analysis outputs of crest horizontal (x-axis) displacement that occurred as a result of the earthquake ground motions combined with the static loads for three load combination cases. As can be seen, the dynamic effects of the OBE and SEE ground motions have caused significant amplification of the PGAs to a greater extent. Similar to the displacement responses, the acceleration responses of both types of dams have more or less similar trends of increase in dynamic amplifications and their magnitudes in the event of the corresponding earthquakes. The dynamic amplification effects of the OBE and SEE ground motions in reservoir full conditions (i.e. load combinations-1 and-2) is comparatively much higher than the corresponding effects that occurred with OBE ground motion in empty reservoir condition (i.e load combination-1) in both types of dams.

Comparison of crest horizontal acceleration values indicates that both types of dams have similar trends of response quantities under the three earthquake load combination cases as described in Table 4-2 below.

Table 4-2 Directional (x-axis) accelerations at crest level under OBE and SEE ground motions

Loading condition	Earthquake ground motion	Reservoir condition	Directional acceleration (x-axis)		Reference
			RCC	CVC	
Load comb.1	OBE in upstream direction	Empty	12.69 m/s ² (1.29g) at time step 0.27 sec.	12.67m/s ² (1.29g) at 0.26 sec.	Figure 4-4 (a to d)
Load comb.2	OBE in downstream direction	Full (normal pool)	25.48 m/s ² (2.60g) at time step 0.35 sec.	28.70 m/s2 (2.93g) at 0.335 sec.	Figure 4-5 (a to d)
Load comb.3	SEE in downstream direction	Full (normal pool)	25.59 m/s ² (2.61g) at time step 0.35 sec.	28.90 m/s2 (2.95g) at 0.34 sec.	Figure 4-6 (a to d)

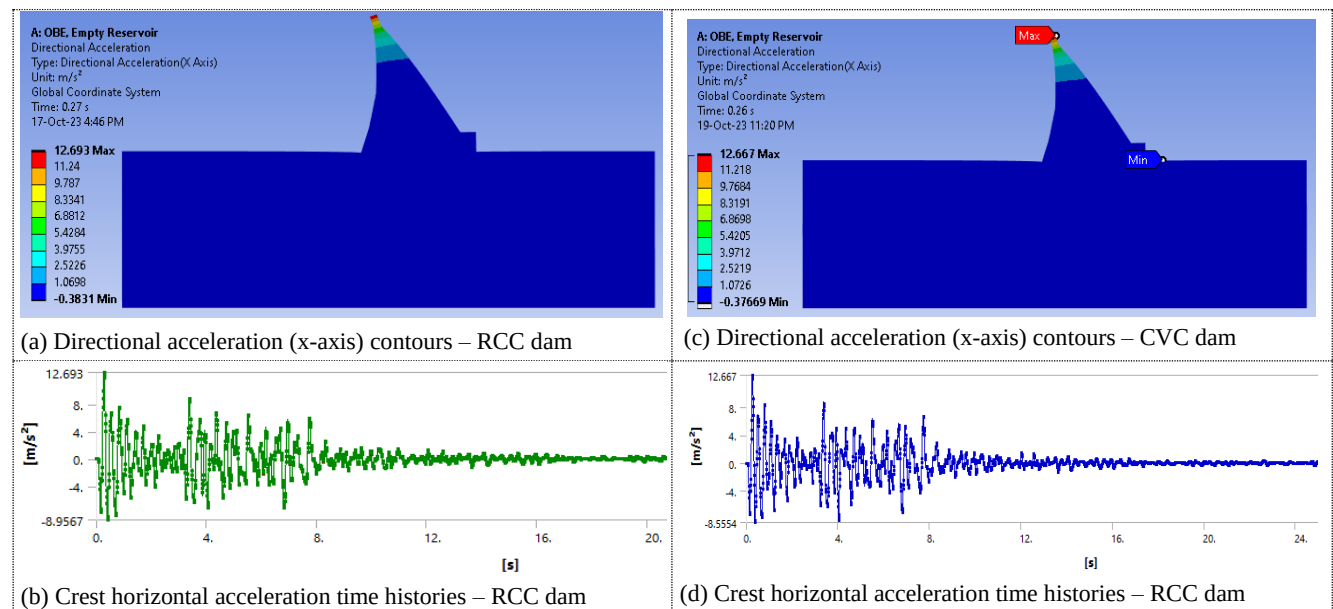


Figure 4-4 Directional (horizontal) accelerations: OBE loading case, empty reservoir condition

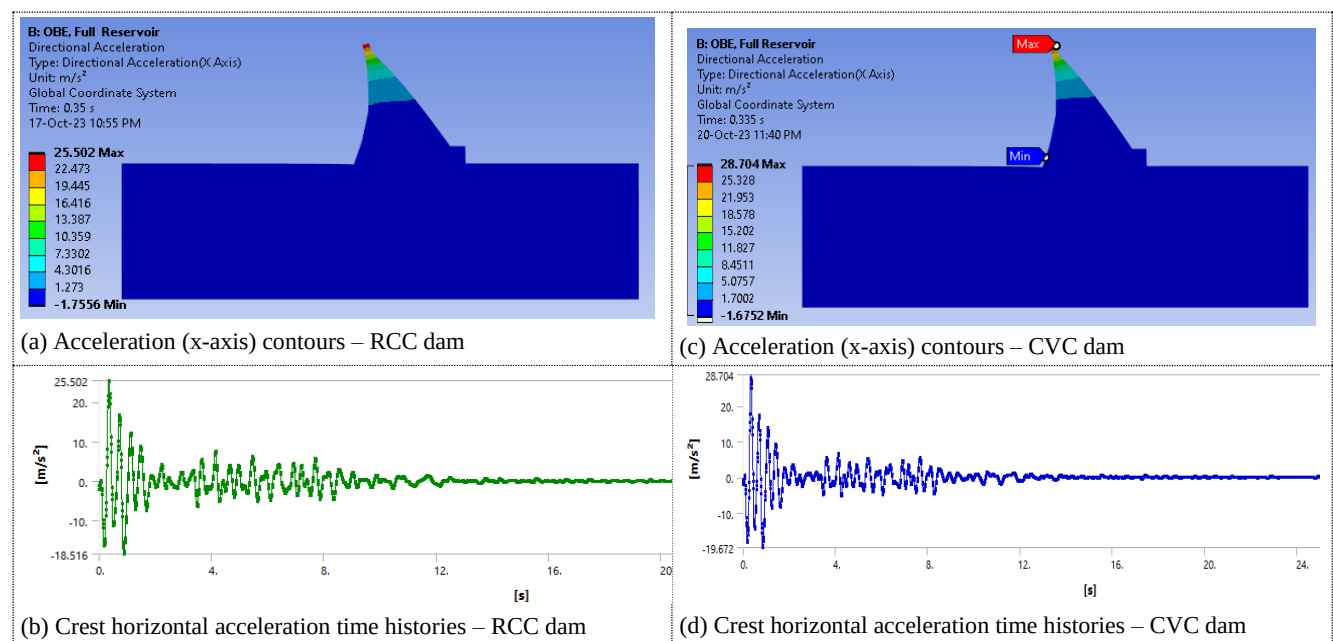


Figure 4-5 Directional (horizontal) accelerations: OBE loading case, full reservoir condition

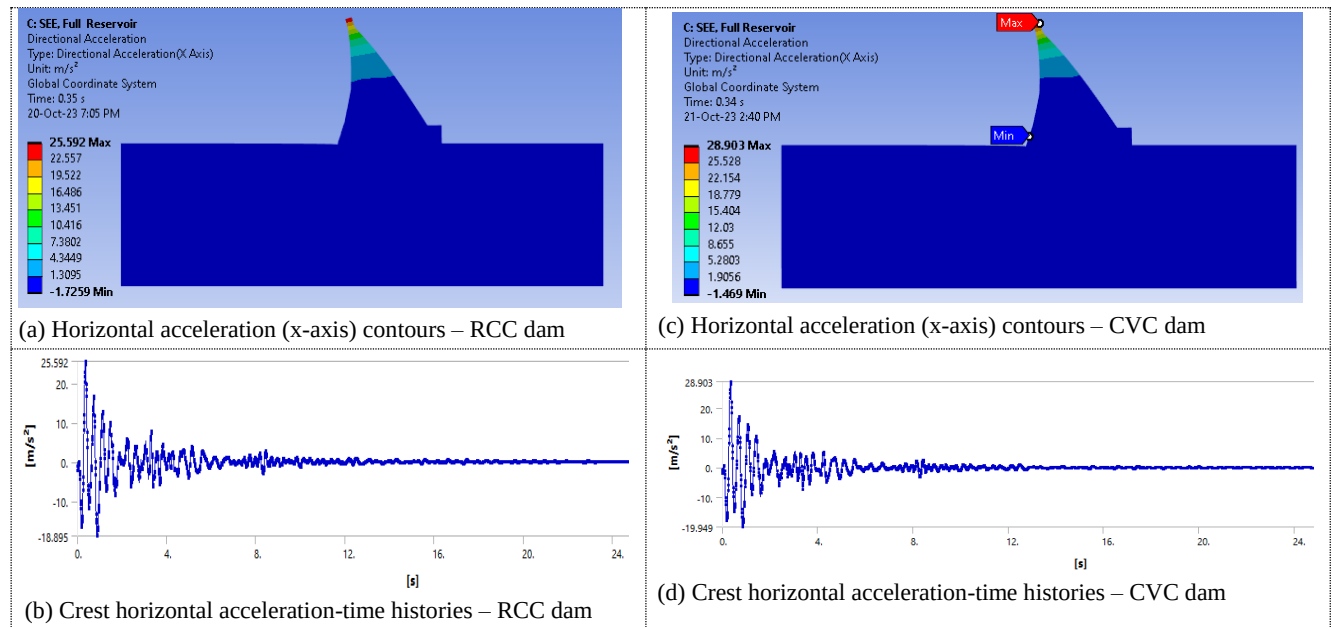


Figure 4-6 Directional (horizontal) accelerations: SEE loading case, full reservoir condition

4.4 Maximum Principal (Tensile) Stresses

Table 4-3 summarizes the maximum principal stresses, which are defined as tensile stresses, in both types of dams when subjected to seismic excitations. The maximum tensile stresses occur in different parts of the dam, mainly at the heel, toe, and upper elevations of the downstream slope, and locations of slope changes. In this particular case study, the largest magnitude of tensile stress occurs at upper elevations of the dam's downstream slope (Figure 4-7 (a) and (c)). Comparison of the maximum tensile stresses between the two types of dams demonstrates that the DCR values computed for responses of both types of dams are very close to each other, and can be regarded as having “similar response quantities” due to both types of dams having the same class of concrete (compressive strength 18MPa) on the outer regions which are subject to large magnitude of tensile stresses. The stress demands associated with the DCR values of 1 and 2 were calculated according to the formula developed by Raphael (1984), as cited in USACE 2003/2007, using the following equations.

The stress demand for DCR=1 is calculated by: $f'_t = 0.32 * f'_c^{2/3} = 2.2 \text{ MPa}$

The stress demand for DCR=2 is calculated by: $f'_t = 0.64 * f'_c^{2/3} = 4.4 \text{ MPa}$ (used $f'_c = 18 \text{ MPa}$)

In the RCC dam, the highly stressed region is the exterior region (RCC5) that has a compressive strength of 18MPa. Similarly, in the CVC dam, the entire section of the concrete has uniform compressive strength of 18MPa. The stress demand corresponding to DCR=1 is calculated as 2.20MPa, and that to DCR=2 is 4.40MPa for both types of dams.

Table 4-3 Maximum principal (tensile) stresses under OBE and SEE ground motions

Loading condition	Earthquake ground motion	Reservoir condition	Maximum principal (tensile) stresses		DCR		Reference
			RCC	CVC	RCC	CVC	
Load comb.1	OBE in upstream direction	Empty	2.30 MPa at 0.28sec. (d/s slope, upper slope elevations); 0.418MPa at 0.37 sec. (at heel)	2.18 MPa at 0.27sec. (d/s slope, upper elevations) 0.437 MPa at 0.39 sec. (at heel)	2.30/2.20=1.04 0.418/2.20=0.19	2.18/2.20=0.99 0.437/2.20=0.20	Figure 4-10 (a to d)
Load comb.2	OBE in downstream direction	Full (normal pool)	7.86 MPa at 0.355sec. (d/s slope, upper elevations) 4.55 MPa at 0.45sec. (at heel)	7.33 MPa at 0.34 sec. (d/s slope, upper elevations); 4.50 MPa at 0.44 sec. (at heel)	7.86/2.20=3.57 4.55/2.20=2.07	7.33/2.20=3.33 4.58/2.20=2.08	Figure 4-11 (a to d)
Load comb.3	SEE in downstream direction	Full (normal pool)	7.87 MPa at 0.355sec. (d/s slope, upper elevations); 4.52 MPa at 0.11sec. (at heel)	7.34 MPa at 0.34 sec. (d/s slope, upper elevations); 4.38 MPa @ 0.12 sec. (at heel)	7.87/2.20=3.57 4.52/2.20=2.05	7.34/2.20=3.34 4.38/2.20=1.99	Figure 4-12 (a to d)

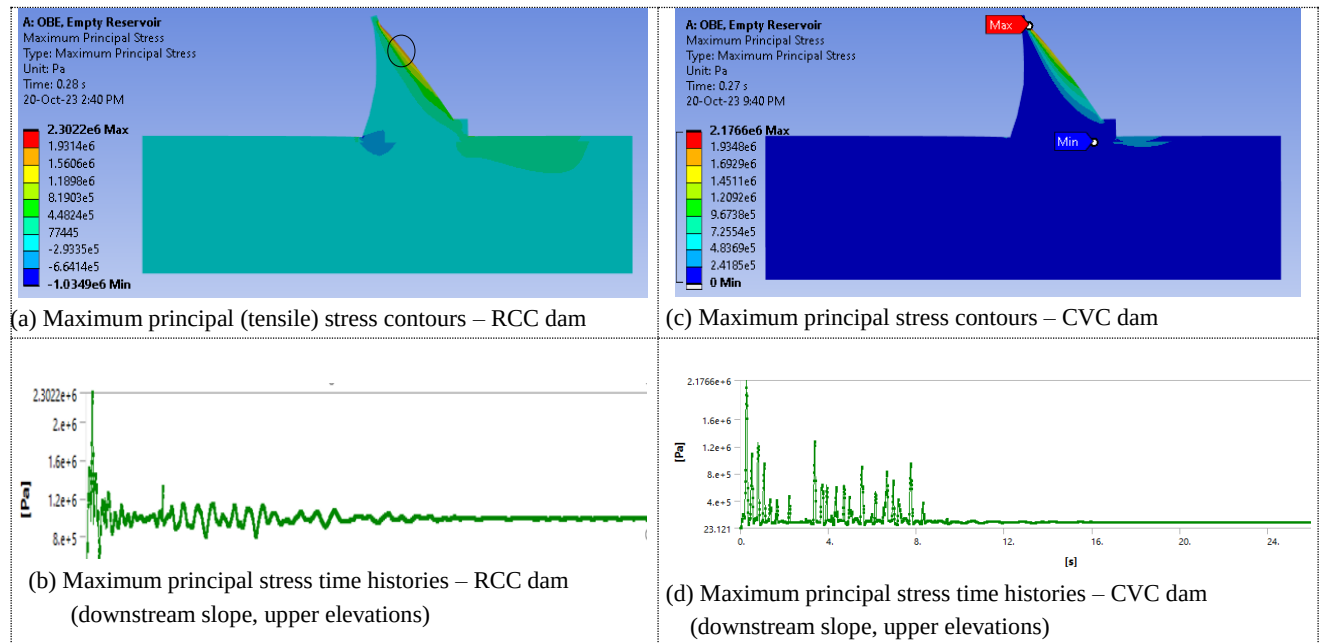


Figure 4-7 Maximum principal (tensile) stresses: OBE loading case, empty reservoir condition

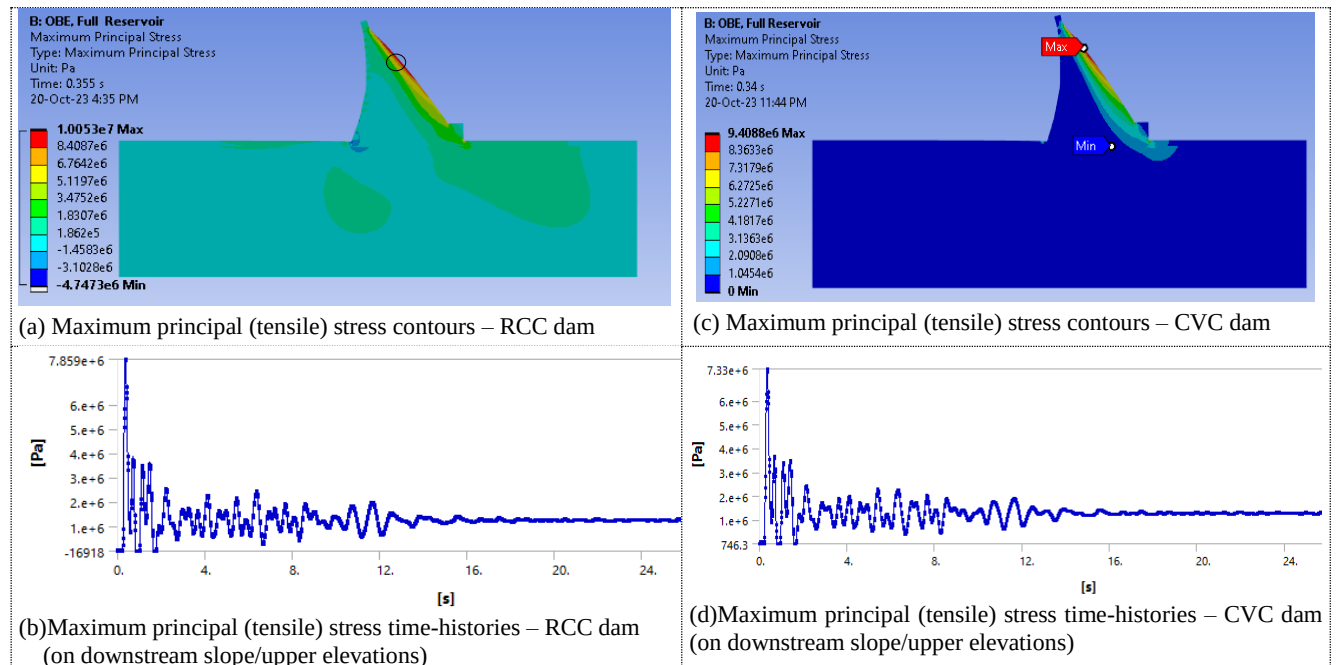


Figure 4-8 Maximum principal (tensile) stresses: OBE loading, full reservoir condition

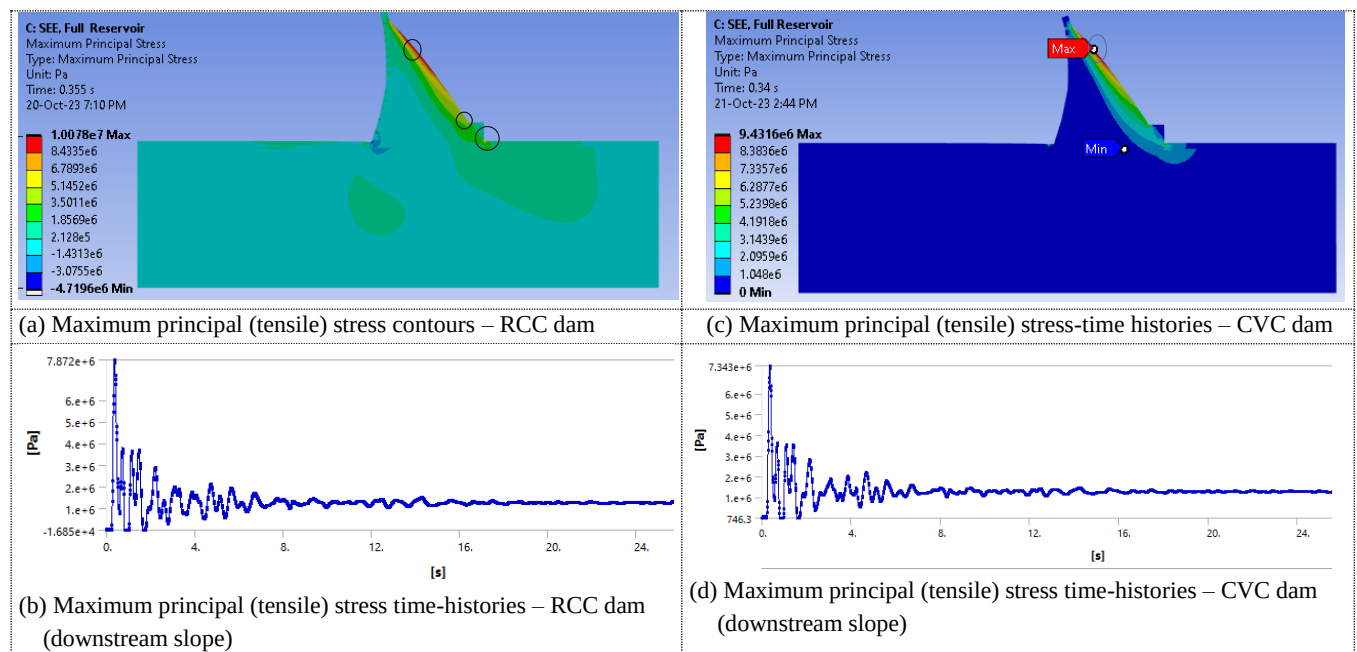


Figure 4-9 Maximum principal (tensile) stresses: SEE loading, full reservoir condition

4.5 Maximum Shear Stresses

Table 4-4 summarizes the analysis outputs of maximum shear stresses in both types of dams when subjected to seismic excitations under the three load combination cases. The maximum shear stresses occur at the bottom heel points in both types of dams. The computed shear stress values indicate that the maximum shear stress values in both types of dams are very close to each other under all load combination cases. However, the normal stresses at the heel points have shown tremendous increase in magnitude associated with the reservoir full conditions under load combinations-2 and-3.

The computation of factors of safety against shear stresses, illustrated in Table 4-5, and comparison of the corresponding results for the two types of dams, shows that the two types of dams have similar trends of response quantities in terms of seismic performance against shear/sliding resistance.

Table 4-4 Maximum shear stresses

Loading condition	Earthquake ground motion	Reservoir condition	Maximum shear stress, $\tau_{\max}$ (MPa)		Reference
			RCC	CVC	
Load comb.1	OBE in upstream direction	Empty	5.62MPa at 0.415sec. (at heel)	5.75 MPa at 0.39 sec. (at heel)	Figure 4-10 (a) and (b)
Load comb.2	OBE in downstream direction	Full (normal pool)	29.27 MPa at 0.52 sec. (at heel)	30.79 MPa at 0.50 sec. (at heel)	Figure 4-11 (a) and (b)
Load comb.3	SEE in downstream direction	Full (normal pool)	29.94 MPa at 0.52sec. (at heel)	31.21 MPa at 0.50 sec. (at heel)	Figure 4-12 (a) and (b)

Table 4-5 Factor of safety against shear stresses

Loading condition	Earthquake ground motion	Reservoir condition	Normal stress, σ_n (MPa)		Shear strength (MPa) $\tau_c = c' + \sigma_n \tan \phi'$		Factor of safety, $FS = \tau_c / \tau_{\max}$	
			RCC	CVC	RCC	CVC	RCC	CVC
Load comb.1	OBE in upstream direction	Empty	8.13	8.22	9.33	9.42	9.33/5.62=1.62	9.42/5.75=1.64
Load comb.2	OBE in downstream direction	Full (normal pool)	30.57	32.96	31.77	34.16	31.77/29.27=1.10	34.16/30.79=1.11
Load comb.3	SEE in downstream direction	Full (normal pool)	31.20	32.86	32.40	34.06	31.20/29.94=1.04	34.06/31.21=1.09

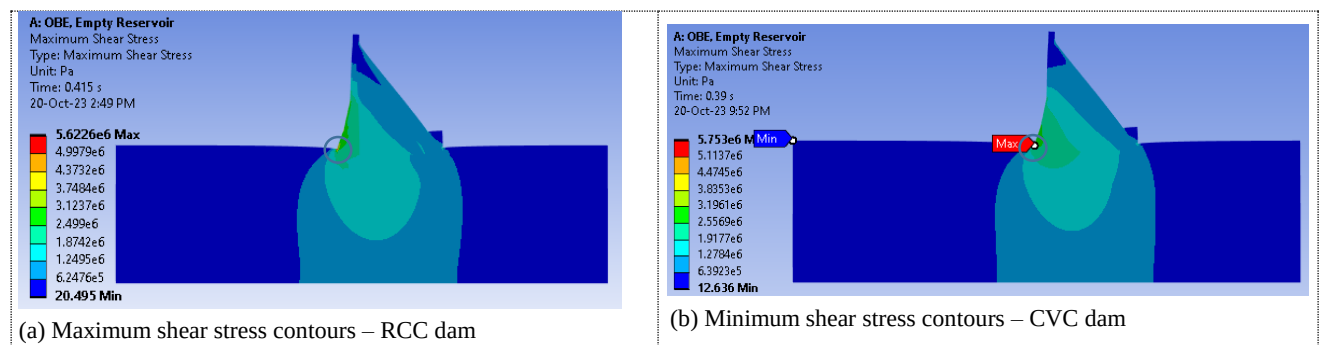


Figure 4-10 Maximum shear stresses: OBE loading case, empty reservoir condition

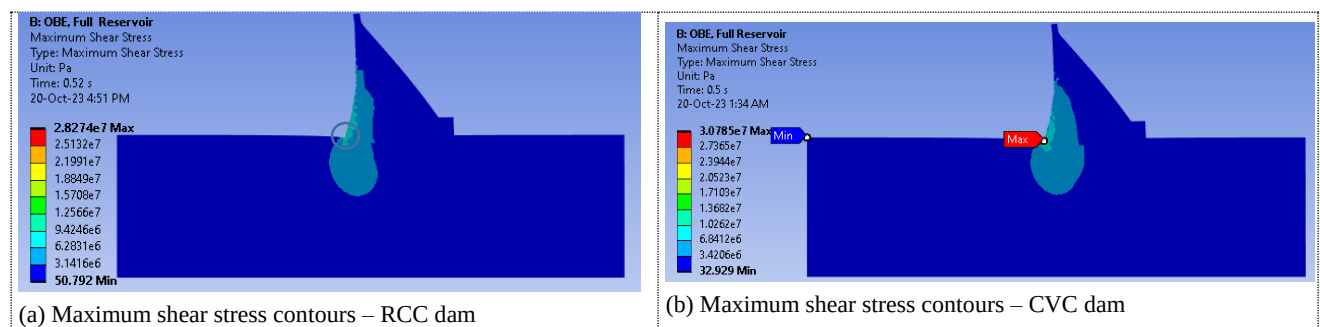


Figure 4-11 Maximum shear stresses: OBE loading case, full reservoir condition

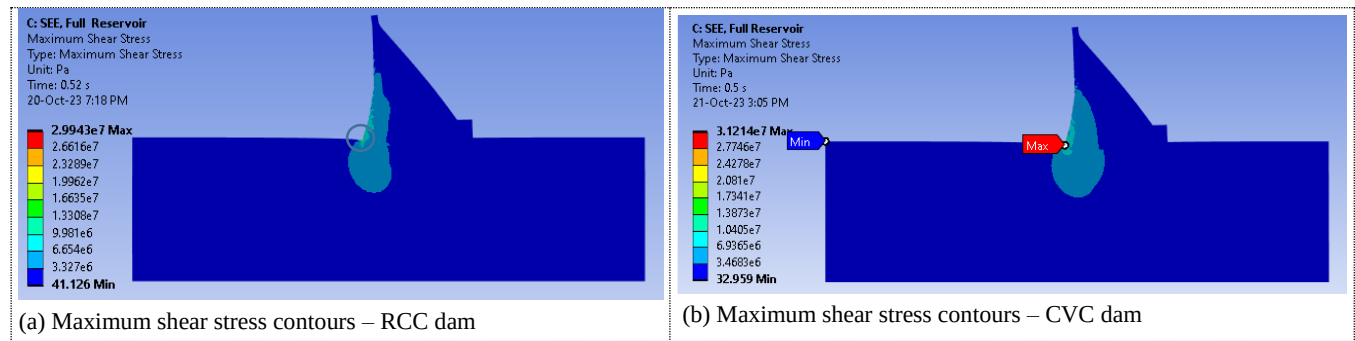


Figure 4-12 Maximum shear stresses: SEE loading case, full reservoir condition

4.6 Comparative Seismic Performance Evaluation/Rating Per USACE (2003/2007) Guidelines

The comparative seismic performance evaluation between the RCC and CVC types of gravity dams was carried out in accordance with the performance /damage acceptance criteria set for linear-elastic time history analysis for concrete gravity dams provided in the USACE (2003, 2007) and Ghanaat's (2014) guidelines.

The seismic response evaluation of both types of dams was based on the use of the performance/damage rating curve which requires computation of Demand-Capacity Ratios (DCRs) and the cumulative inelastic duration of stress excursions exceeding the tensile strength of the concrete on critical sections of the dam. In the performance evaluation/damage rating, the time-histories of maximum principal (tensile) stresses in the highly overstressed regions of the dams were used to assess the severity of the stress effects and level of anticipated damages because of the fact that the tensile strength requirement is generally thought as the governing criteria that controls the seismic design/performance of concrete gravity dams. Three categories of comparisons, associated with critical earthquake loading conditions listed below, were made between the two types of dams:

- 1) RCC vs CVC responses: OBE ground motion in empty reservoir condition
- 2) RCC vs CVC responses: OBE ground motion in full reservoir condition
- 3) RCC vs CVC responses: SEE ground motion in full reservoir condition

4.6.1 RCC vs CVC Responses: OBE Ground Motion, Empty Reservoir Condition

Figure 4-13 below depicts the time history of maximum principal (tensile) stresses at critical sections (in this study, the downstream slope at mid and upper elevations) of both types of dams when subjected to seismic excitations under the OBE ground motion in empty reservoir condition. This figure shows that the stress values in both types of dams are well below the stress demands for DCR=1 (i.e 2.20 MPa) except at the initial time step of 0.27 and 0.28 seconds where the computed tensile stresses are close to / slightly higher than 2.2MPa, indicating that there is “no damage” or “acceptable level of damage” due to the OBE loading case in reservoir empty condition in both types of dams.

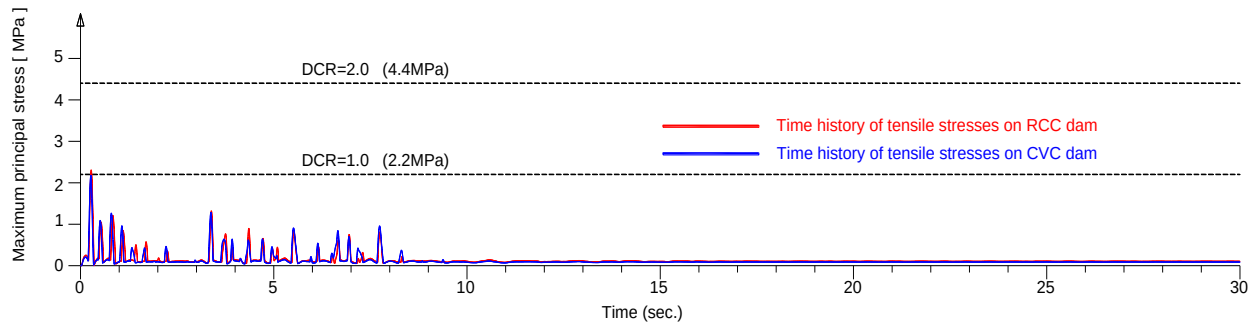


Figure 4-13 Time histories of maximum principal (tensile) stresses on RCC and CVC dam sections (OBE ground motion, empty reservoir condition)

Figure 4-14 below compares the results of linear-elastic time-history analyses for the two types of dams with the acceptance limit set for concrete gravity type of dams (USACE 2003/2007). The computed DCR values for both types of dams are close to unity (one). It is clearly understood in this figure that the cumulative inelastic duration of stress cycles at the downstream slopes of both types of dams are well below the acceptance limit set for the linear-elastic analysis, indicating “low to moderate damage” in both types of dams. In the case study, however, the idealized CVC dam type demonstrated slightly better seismic performance rating than the RCC dam type.

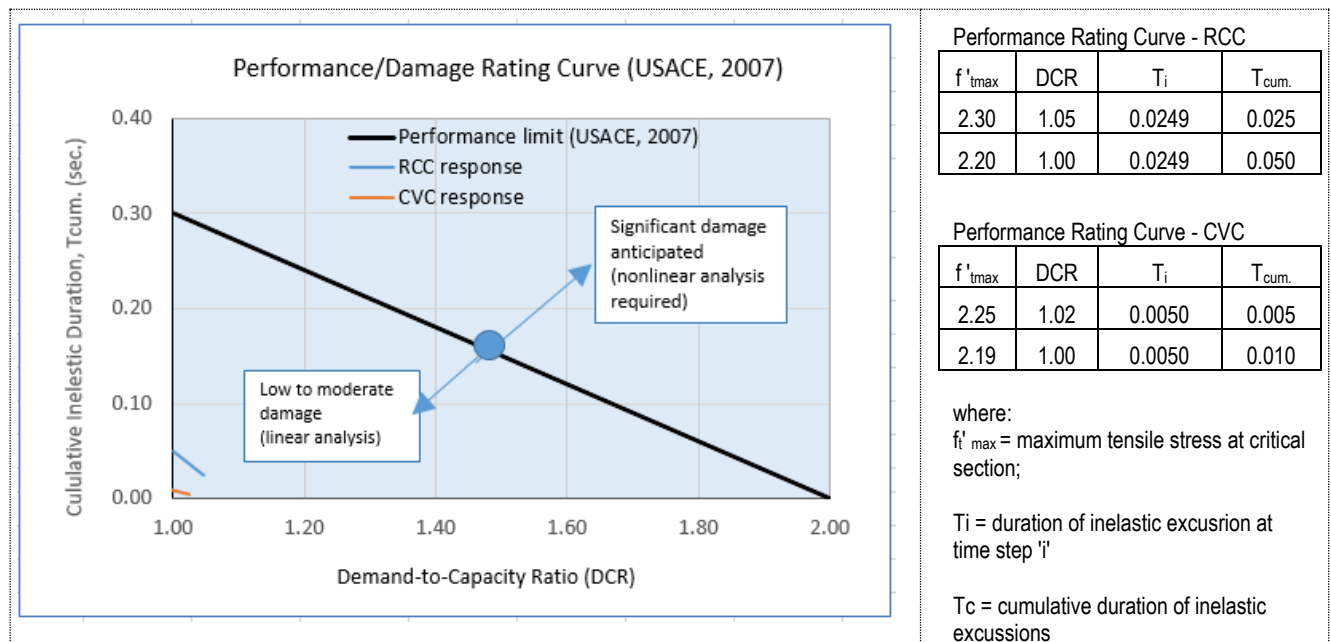


Figure 4-14 Comparison of cumulative duration of stress cycles with acceptance limits for maximum tensile stress cycles (OBE ground motion, empty reservoir condition)

4.6.2 RCC vs CVC Dams: OBE Ground Motion, Full Reservoir Condition

Similar to the earthquake load combination case with OBE event in empty reservoir condition, Figure 4-15 below illustrates the envelopes of maximum principal (tensile) stresses at critical sections (in this study, the downstream slope at mid and upper elevations) of both types of dams when subjected to the OBE ground motion in full reservoir condition. Both types of dams can be regarded as having similar response behaviors under OBE ground motions in reservoir full condition.

The first stress cycles in both types of dams, with DCR being much greater than 2, significantly exceeds the acceptance limit for linear-elastic analysis (DCR=2). The next three stress cycles are falling between the stress demands for DCR= 1 and DCR=2. The stress envelope profile indicates that there is a likelihood of “serious damage” in the life time of the dam, and hence the need for a nonlinear analysis is justifiable to ensure that there is less or no probability of collapse under similar earthquake loading conditions.

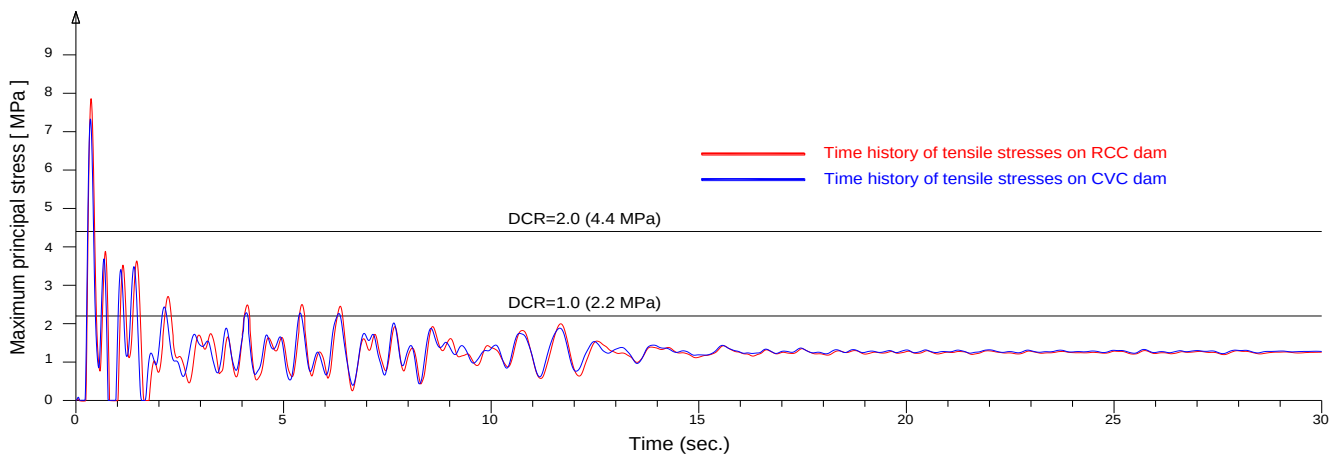


Figure 4-15 Time histories of maximum principal (tensile) stresses on RCC and CVC dam sections (OBE ground motion, full reservoir condition)

The performance rating curves (Figure 4-16 below) illustrate the comparison of the linear-elastic analysis responses between the two types of dams under OBE/full reservoir condition. The computed DCR values for eight stress cycles in both types of dams are much greater than one (i.e. between 1 and 4). It can be noted that the cumulative inelastic duration of stress cycles at the downstream slopes of both types of dams are significantly above the acceptance limit set for the linear-elastic analysis, indicating the probability of “serious damage”, which suggests that nonlinear time-history analyses are required to estimate more accurately of the most probable level of damage and ensure “no collapse” would occur. In the case study, both types of dam demonstrated similar performance/response behaviours against the OBE event in full reservoir condition.

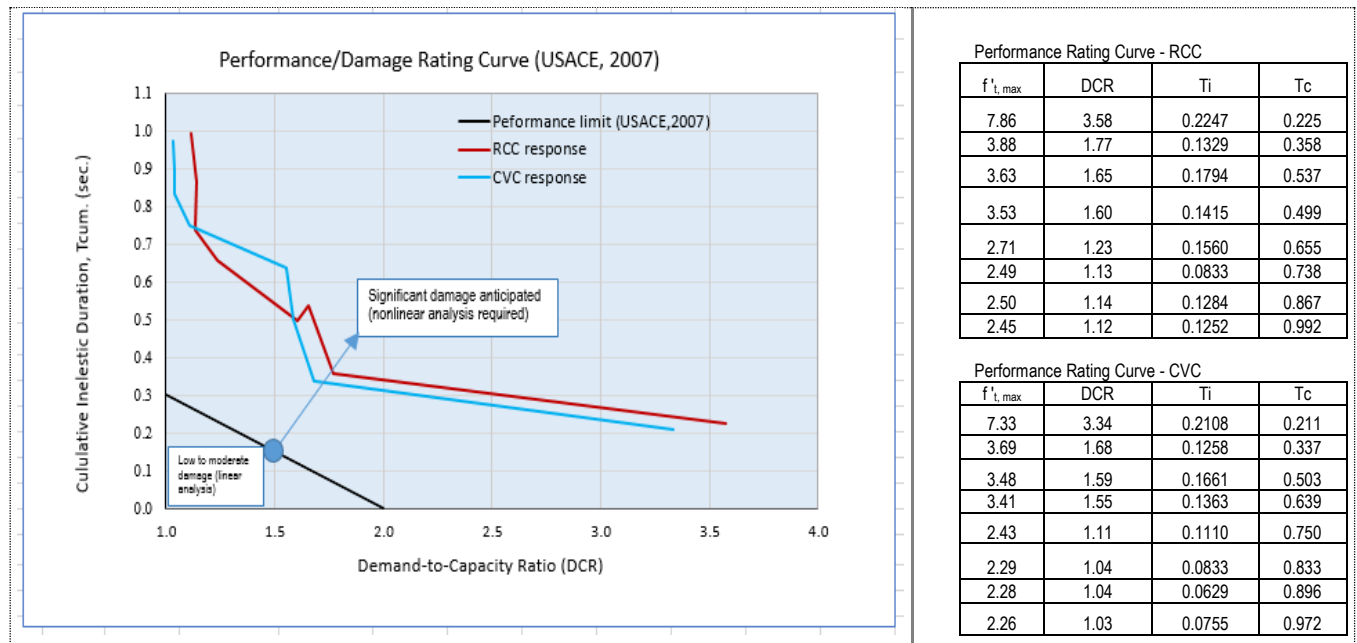


Figure 4-16 Comparison of cumulative duration of stress cycles with acceptance limits for maximum tensile stresses (OBE ground motion, full reservoir condition)

4.6.3 RCC vs CVC Dams: SEE Ground Motion, Full Reservoir Condition

Figure 4-17 below illustrates the envelopes of maximum principal (tensile) stresses at critical sections (in this study, the downstream slope at mid and upper elevations) of both types of dams when subjected to the SEE ground motion in full reservoir condition. Both types of dams can be regarded as having similar response behaviours under such earthquake loading condition.

The first stress cycles in both types of dams, with DCR being much greater than 2, significantly exceeds the acceptance limit for linear-elastic analysis (DCR=2). The next four stress cycles are falling between the stress demands for DCR= 1 and DCR=2. The overall profile of the stress envelope indicates that there is a likelihood of “serious damage” in the life time of the dam, and hence the need for a nonlinear analysis is justifiable to ensure that there is less or no probability of collapse under similar earthquake loading conditions.

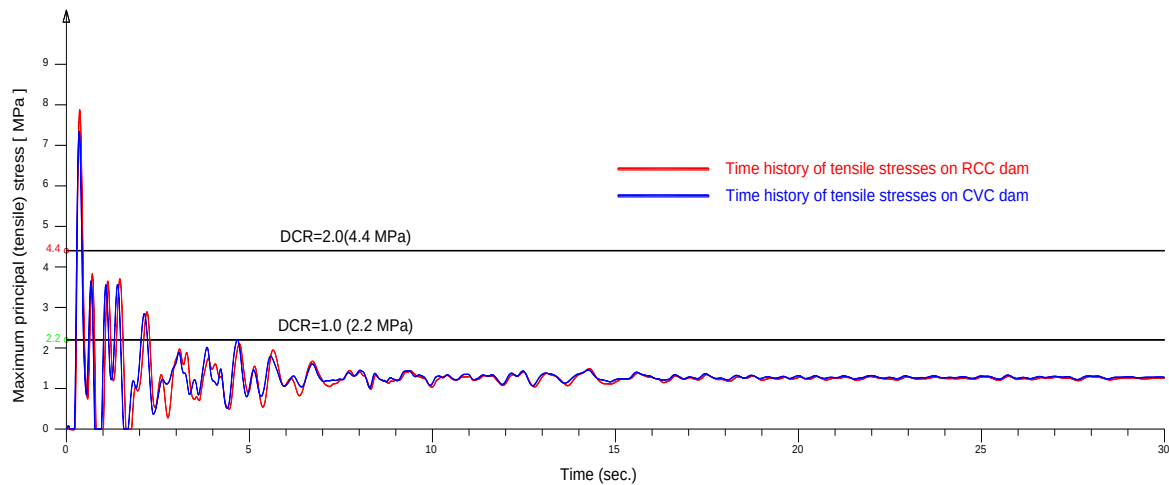


Figure 4-17 Time histories of maximum principal (tensile) stresses on RCC and CVC dam sections (SEE ground motion, full reservoir condition)

The performance rating curves (Figure 4-16 below) illustrate the comparison of the linear-elastic analysis responses between the two types of dams under SEE/full reservoir condition. The computed DCR values for the first stress cycles in both types of dams are much greater than 2, which is above the margin of safety to ensure “damage control performance based on linear-elastic analysis”, and indicates the damage is deemed “severe” and classified as “serious damage” which should, according to the USACE (2003, 2007) guidance, be dealt with using nonlinear analysis approach.

Additionally, it can be noted that the cumulative inelastic duration of stress cycles during these stress cycles are significantly above the acceptance limit set for the linear-elastic analysis, indicating the probability of “serious damage” in both types of dams, which suggests that nonlinear time-history analyses are required to estimate more accurately the most probable level of damage and ensure “no collapse” would occur. In the case study, both types of dam demonstrated similar performance/response behaviors against the SEE event in full reservoir condition.

The overall performance rating clearly demonstrates that both RCC and CVC dams, if they have similar geometry and concrete qualities, would demonstrate similar seismic performance/damage control behaviours in the event of SEE/reservoir full condition.

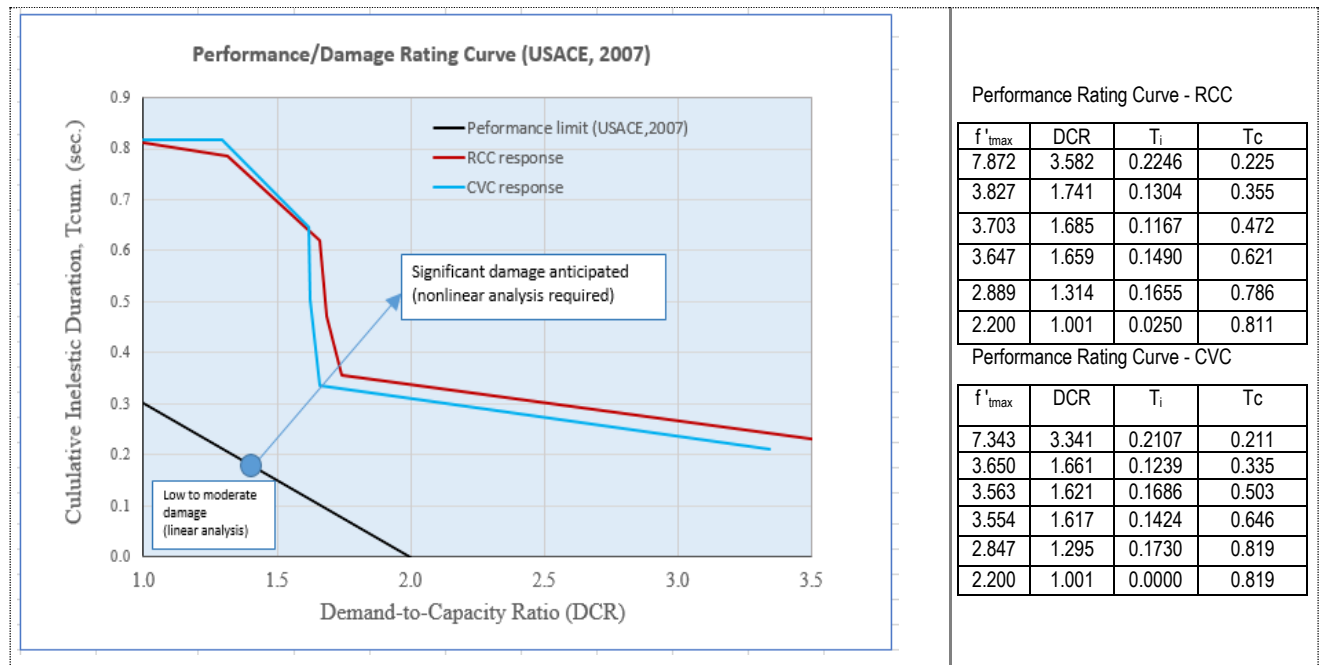


Figure 4-18 Comparison of cumulative duration of stress cycles with acceptance limits for maximum tensile stress cycles (SEE ground motion, full reservoir condition)

4.7 Discussions

The purpose of this study is to demonstrate the application of USACE (2003, 2007) and related manual guidelines in the seismic response analysis and comparison of performances of the two major types of concrete gravity dams (RCC and CVC types) under earthquake loading conditions. The two types of dams were considered to have equivalent sizes and dimension details except that the RCC dam is an existing dam while the CVC type is an idealized dam developed with recommended material properties for comparative analysis in this study. The two types of dams were analyzed using linear-elastic time-history analysis using spectral-matched acceleration-time histories. The response quantities of both types of dams were further checked for compliance with performance/acceptance criteria set for the linear elastic range as provided in the USACE (2003, 2007) guidelines. The evaluation was based on the determination of the DCR values at critical sections of the dam (in this case study, on the downstream slope faces) and estimation of the cumulative inelastic durations of stress excursions in the overstressed regions of the entire concrete body of the dams.

The linear time history analyses was done using modern FEA approach with the help of finite element simulation software ANSYS workbench 2023. The analysis results include crest maximum displacements (directional deformations) with time-histories, crest peak horizontal accelerations with time histories, envelopes (contour bands) of maximum principal stresses and shear stresses. The ANSYS software outputs of envelopes of maximum principal (tensile) stresses were used to investigate the extent and severity of the highly overstressed regions of the dam under three loading conditions (OBE/empty reservoir, OBR/full reservoir, and SEE/full reservoir conditions). Demand-to-Capacity Ratios (DCRs) were calculated to investigate the differences/similarities in performance

levels of the two types of dams when subjected to similar earthquake loading conditions combined with the usually occurring static loads. Finally, the seismic response quantities of the two types of dams were compared and evaluated using the performance criteria (thresholds) provided in the USACE (2003, 2007) guidelines, which were discussed in the previous sections.

The general trends of analysis results for the empty and full reservoir condition show that the dam-reservoir-foundation system generally exhibits an overall increase in the response quantities of crest displacements and horizontal accelerations associated with the full reservoir conditions in comparison with the corresponding response quantities associated with the empty reservoir condition. In particular, the crest displacements, crest horizontal accelerations, and maximum tensile stress responses represent a substantial increase in full reservoir conditions when compared with the response quantities estimated in the empty reservoir condition. It is only with OBE ground motion in empty reservoir condition that both types of dams showed DCR values and cumulative inelastic durations falling inside the performance limit set for linear-elastic analysis. In the event of both OBE and SEE ground motions in full reservoir conditions, both types of dams exhibited similar trends of response behaviours and the cumulative inelastic durations are falling above the performance limit set for linear-elastic analysis, which indicates that they should be analyzed using nonlinear analysis approaches for a more realistic and accurate estimation of the level of damages that will consider nonlinear material behaviors, cracks and joint openings in the body of the dam.

From the summary of the linear time history analysis results presented in section 4.2 through to 4.5, and the comparative seismic performance evaluation / damage rating calculations presented in section 4.6, we can simply come to a general understanding that the two types of dams have similar trends of response behaviours when subjected to similar earthquake load combination cases. The similarity exists in response quantities, including displacements, accelerations, tensile stresses, and shear stresses.

Despite their similarities in seismic response behaviors to a great extent, the response quantities (especially the DCR values) are found to be highly dependent on the grade of concrete, primarily of the compressive strength and modulus of elasticity, used in the highly overstressed exterior regions of the dam such as the heel, toe and downstream sloping faces. A higher grade of concrete provides a relatively higher modulus of elasticity and tensile strength which in turn results in lesser deformations and lesser DCR values, indicating better performance rating. Many of the previously constructed existing CVC dams had uniform and homogenous mixes of higher grades of concrete and aggregate quality, as opposed to the RCC construction which allows the use of lower grades of concrete in different zones of the dam, moderate to low quality of aggregates, and the existence of greater number of horizontal lift joints that resulted in reduced tensile strength values. In addition, the deformations induced by the earthquake to the idealized CVC dam is slightly less than the

corresponding value for RCC dam, which could be associated to the homogenous concrete mix with higher modulus of elasticity contributing to this situation.

The performance evaluation/rating that indicates the need for nonlinear 2-D or 3-D analysis in both RCC and CVC dams, suggest that the cracking would initiate on the mid-upper elevations of the downstream sloped face of the dam and the cracks may propagate throughout the entire downstream sloped surface of the dam. Hence, the nonlinear time-history analysis is needed to further evaluate the performance evaluation or damage rating and the overall effects on the dam stability in general.

4.8 Validation of Results

The basic concept of validation of analysis results in any research lies in the fact that it is an important phase that should be done by the researcher to ensure that the analysis results obtained from computational process, based on collection and processing of the required data, achieves the stated objectives of the research. It also includes a process of understanding whether the methodology utilized in the research process is effective enough to ensure that the research problems are effectively answered.

In this study, the validation of analysis results have been linked to the following grounds, as briefly explained below.

1. Data collection from the dam owner

The entire set of data used in this research are realistic data obtained from geotechnical investigations and successive laboratory testing and verifications made at several stages of the project and utilized by the dam owner (Ethiopian Electric Power, EEP) in the process of successive design reviews and implementation of the construction of the existing RCC dam. The material model developed for the idealized CVC dam was also established based on the geotechnical investigations, laboratory test results and material properties of the part of the existing dam already constructed of similar material types and qualities.

2. Previously published seismic studies reviewed

The design/evaluation earthquakes for the intended purpose of the research were established based on comparison of the site-specific seismic hazard assessment carried out by the project and accepted by the dam owner (EEPCo, 2007) with two previously conducted seismic studies: the first is the site-specific seismic hazard study for Gibe III dam site (Aman, A., 2020), and the second one is Africa regional and horn seismic studies (Atalay, A., 2017). The estimated hazard levels, PGAs, used for the design stage of the project (0.05g for OBE and 0.15g for SEE) were found much lower than the values estimated by the other researchers (0.10g for OBE, and 0.34g for SEE). The SEE hazard level used in the design stage, was also found lower than the PGA of 0.18g estimated for 475-year return period by Ayele, A. (2017). In this study, SEE of 0.34g and OBE of 0.10g were considered.

3. Basis of Seismic Performance Evaluation

The seismic performance evaluation and acceptance criteria were established based on the internationally recognized and accepted guidelines provided in USACE (2007), EM 1110-2-6053, and related manuals. These guidelines have been widely used by numerous designers and researchers across the globe in many countries.

4. The use of state of the art practices for dynamic analysis

The seismic performance evaluation was carried out using ANSYS Workbench, Multiphysics component (2023 version), that is currently one of the leading finite element simulation programs that helps in the study of complex interactions between fluid, structural, electromagnetic, thermal and other forces at a global level by many practicing professional engineers and researchers.

5. Comparison with similar researches and results

Several researches were conducted by many people to assess the seismic performance of concrete gravity dams including both RCC and CVC types of dams. Comparison of results with similar researches conducted using linear time history analysis for concrete gravity dams with maximum nonoverflow sections measuring 75 to 120m indicated that the analysis results obtained in this study are quite reasonable and realistic and hence can be accepted as valid results for further reference or updates as necessary.

Table 4-6 Typical examples of linear time history analysis results for concrete gravity dams

Dam name/ year completed/ country	Dam type / height	Earthquake	Linear elastic analysis results	Reference
Koyna dam (1963), India	Concrete gravity dam (straight), 103m high	1967 Koyna Earthquake, M 6.5 PGA @ lower gallery: 0.36-0.63g	Maximum principal stress: 6.2MPa (downstream face at slope change)	USSD 2017; USCOLD 1992
Folsom dam (1956), USA	Concrete gravity dam, 82.45m high	PGA 0.3g	Crest displacement: 1.59cm at empty reservoir condition; 2.59cm at full reservoir condition; Peak crest acceleration: 2.45g at empty reservoir 2.59g at full reservoir; Maximum tensile stress: 4.62MPa at empty reservoir; 6.77MPa at full reservoir	USACE (2003), EM 1110-2-6051
Andiraz dam , Turkey	RCC dam, height 141m	OBE= 0.20g, MDE =0.30g, MCE =0.65g	Peak crest displacement: 18cm at MCE; Peak crest acceleration: 2.1g at MCE; Maximum tensile stress: 3.5MPa at OBE; 3.6MPa at MDE, 5.0MPa at MCE	Aldemir, A., Akpınar, U., Arici, Y., & Binici, B. (2012)

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

This thesis presents the comparative seismic performance between the two most common types of concrete gravity dams – RCC and CVC dams – using the linear-elastic time history analysis, according to the USACE (2007, 2003) guidance. To perform this, Gibe III dam was selected as a case study and the earthquake response behaviors of the existing dam was compared against that of an idealized CVC dam with equivalent size and material properties. Previous studies of site-specific seismic hazard assessment for the dam site were referred to estimate the peak ground accelerations (PGAs) for the design/evaluation earthquakes comprising the safety evaluation earthquake (SEE) and operating basis earthquake (OBE) ground motions, as recommended by the ICOLD (2010). The target response spectra were anchored to the standard spectral models defined in the Ethiopian code ES EN 1998: 2015. About four ground motion acceleration-time histories were selected from PEER-NGA database that are representative of the design ground motions, specified in terms of a target response spectrum and the desired characteristics of the earthquake ground motions in terms of earthquake magnitude, source-to-site distance, fault mechanisms and other general site characteristics. The selected ground motions were scaled using SeismoMatch software to match the characteristics of the desired target response spectra.

The seismic performance of the an existing RCC dam and an idealized CVC dam was evaluated following the US Army Corps of Engineers (2007) guidelines, using an advanced state of the practice simulation and finite element program – ANSYS Mechanical 2023. The finite element models of the two dams were developed using quadrilateral (four-node) and triangular (three-node) elements. Scaled ground motion acceleration-time histories were applied as seismic loads to the finite element models of the dams. The seismic response analyses were made by considering appropriate load combinations comprising OBE and SEE ground motions combined with static loads. Dynamic analysis methodologies were properly followed to determine the seismic responses of equivalent sizes of both dam types – RCC and CVC dams. The results were then evaluated and compared against each other for both dam types with reference to the acceptance criteria set for linear-elastic time history analysis provided in the USACE (2003, 2007) guidance.

Based on the analyses of the results of linear analyses, it is understood that very high tensile stresses have developed at the bottom of heel, lower elevations of upstream dam face, at toe of dam, and at downstream slope/upper elevations near the change of slopes. Both types of dams were found to be safe against serviceability loading conditions in the linear elastic range when subjected to OBE ground motion events in empty reservoir condition. However, in the extreme earthquake load combination cases (OBE ground motions in reservoir full condition, and SEE in reservoir full condition) considered in the analysis, the maximum tensile stresses obtained from linear analysis are

much higher than the apparent tensile strength of concrete, and the cumulative inelastic durations are also falling above the performance limit set for linear elastic analysis, indicating the likelihood of “serious damages” to occur in critical sections, and thereby the need for nonlinear analysis with 2-D and/or 3-D structural modeling to ensure that the dams are safe against potential damages causing possibility of failures or collapse.

According to step-by-step comparisons made for each critical evaluation parameters, and the performance evaluation made using the USACE (2003. 2007) guidelines, it is possible to conclude that both types of dams have similar seismic response behaviour in deformations and stresses unless there exists significant differences in terms of the quality and strength properties of materials (like compressive strength, tensile strength, modulus of elasticity) that would possibly bring differences in their response behaviors. Performance evaluation using the USACE (2007) guidelines indicates that the DCR values are greater than two in the event that the dams are exposed to both OBE and SEE ground motion in reservoir full conditions.

Therefore, both types of dams have comparatively more or less similar seismic response or performance behaviours. In the case study, both types of dams have similar grade of concrete used in the exterior overstressed regions of the dams. The researcher suggests that, if geometric profiles are kept the same for both types of dams, the use of a higher or lower grade of concrete in the dam exterior regions and the type of bedding treatments for lift joints brings a significant change in the tensile strength of the concrete materials and, correspondingly, in their seismic response behaviors.

5.2 Recommendations

It is well known that concrete dams are comparatively less prone to aging, material deterioration, seepage, and have high level of resistance against seismic ground shaking than other embankment types of dams. The fact that well-designed dams have survived long, or there is no significant number of collapse cases in history, does not mean that the dams will remain safe against seismic loads at any time in the future. The importance of periodically re-evaluating the seismic safety of large concrete gravity dams using sufficient design details, seismic analyses methods, updated seismic design/evaluation criteria is unquestionable as seismic analyses methods are still in the process of development up-to-date.

Considering the above stated facts, the following recommendations are drawn from this study:

1) Seismic Response Analysis and Performance Behaviours

The seismic design/evaluation of both RCC and CVC dams is carried out using similar analysis methods. Both RCC and CVC types of dams have similar trends of response behaviors against earthquake events. The only difference could arise from the material models comprising the strength and quality of concrete, the type and nature of bonding between lifts that may cause

reduction in shear and tensile strength of lift joints. It is recommended that the choice between the two types of dams depends upon other governing factors, such as construction method, economy, availability of materials, etc, rather than their seismic response behaviours.

2) The need for nonlinear analysis using 2-D and 3-D finite element modelling

The existing Gibe III dam has a total length of 620m, a maximum nonoverflow height of 236m, and is located in a narrow deep valley with steep side slopes. According to the dam's seismic performance evaluation, presented in section 4.6, the responses of both types of dams to OBE ground motion in reservoir empty condition are inside the performance limit set for linear-elastic range of behavior with "little or no damage". However, the response of both types of dams to OBE and SEE ground motions in reservoir full conditions are found to exhibit nonlinear responses, and hence the need for nonlinear analysis using 2-D and/or 3-D finite element modelling taking into account the real conditions and the actual three-dimensional behavior of the main structure and its supporting features is justified.

3) The need for periodic seismic performance re-evaluation

The importance of periodic evaluation of seismic safety of large concrete gravity dams shall follow recent developments in seismic analysis methods, seismic design/evaluation criteria is unquestionable as seismic analyses methods are still in the process development up-to-date.

For the case study of Gibe III dam, which is located in a seismically active region in the East African Rift System, there is a large difference between the estimated PGAs used in the design stage and the proposed PGAs utilized in this study. Therefore, it is recommended that the site-specific seismic studies for the dam site be revised and updated; and then a nonlinear analysis be adopted for re-evaluation of the seismic performance of the dam to ensure that there is remote probability of failures/collapse in the event that the dam is tested by an equivalent magnitude of earthquake

4) Recommended type of concrete gravity dam in the context of Ethiopia

Considering the potential advantages (cost, time, methodology), that are possibly gained in large proportions from the RCC method of construction, the choice of RCC method of design and construction is highly recommended in the context of Ethiopia. The RCC method allows the use of wide range of material properties that are unsuitable in the CVC method. On the other hand, most of the existing CVC dams in the past were constructed of higher grades of concrete (upto 40MPa of compressive strength), while the use of variable grades/strengths of concrete in different regions of the dam body has become recent developments in RCC dam building practice, which makes the RCC method one of the most preferred choice among concrete dam types.

5) The need for developing national guidelines for seismic hazard studies and seismic safety/performance evaluations

Many site-specific seismic studies for many civil engineering structures in Ethiopia are being carried out on a subjective approach that primarily depend upon the knowledge and experiences of the professionals involved in the data collection, processing, and formulation of approaches and methods of seismic hazard studies. The Ethiopian code of practice, ES EN 1998: 2015 [Design of structures for earthquake resistance], has been developed to address seismic design of building works and small-scale infrastructure developments for a maximum return period of 475 years only. Large-scale development projects such as dams, and power plants, for which defining high magnitudes of seismic hazard levels and setting minimum design/evaluation requirements are not covered in this code and elsewhere. There are no structured standards and guidelines to serve as benchmarks for the professionals. The case of Gibe III dam is an example in which, according to the seismic study for the project, the estimated hazard levels (PGA 0.05g for OBE, and 0.15g for SEE) were much lower than the corresponding hazard levels suggested by other published researches (Aman, 2020; Atalay, A., 2017). In order that such types of significant shortfalls be rectified and standardized, the establishment of standard guidelines for seismic studies that cover all ranges of civil engineering projects and infrastructure developments is a matter of concern in the near future.

6) Contributions of this research to the national development plan of building dams for multiple purposes

This research outlines a well structured guidelines for 2-D linear-elastic time history analysis of concrete gravity dams in accordance with the internationally recognized methodologies and guidelines provided in USACE (2003, 2007) manuals. Three-dimensional linear or non-linear analysis would be further extended based on this study for designing new dams or evaluating existing dams. This research would serve as an important milestone in the development of basic concepts and principles in the dynamic response analysis guidelines and procedures in the Ethiopian context as Ethiopia is currently in the era of planning, construction and operation of existing similar dams and upcoming projects of similar features to meet the national economic development needs. The general and specific findings and recommendations of this research helps in justifying the fact that the choice between the two types of dams in seismically active regions should primarily depend on other governing factors such as construction method, speed of construction, and economic aspects. In the context of Ethiopia, the practice of building large RCC dams has gained popularity in the last 2 to 3 decades, and it is recommended to adapt the same practice as part of the national development plans for building many more dams of similar types for different purposes.

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7 APPENDICES

APPENDIX-I: PEER Ground Motion Data Base [OBE loading case]

RSN454_MORGAN_GIL337.AT2

APPENDIX-II: PEER Ground Motion Data Base [SEE Loading case]

RSN1633_MANJIL_ABBAR--L.AT2

APPENDIX I: PEER Ground Motion Data Base RSN454_MORGAN_GIL337.AT2	APPENDIX II: PEER Ground Motion Data Base RSN1633_MANJIL_ABBAR--L.AT2
PEER NGA STRONG MOTION DATABASE RECORD Morgan Hill, 4/24/1984, Gilroy - Gavilan Coll., 337 ACCELERATION TIME SERIES IN UNITS OF G NPTS= 5999, DT= .0050 SEC, -.9586174E-05 -.9128954E-05 -.8644874E-05 -.8117484E-05 -.7540784E-05 -.6882243E-05 -.6126652E-05 -.5314541E-05 -.4486699E-05 -.3578226E-05 -.2509924E-05 -.1221160E-05 .2221334E-06 .1395988E-05 .1801523E-05 .2235709E-05 .1986435E-05 .6272622E-06 -.9675598E-06 -.3853521E-05 -.9645881E-05 -.1453178E-04 .1326384E-04 -.6781537E-05 .6650837E-05 .2210731E-04 .2218358E-04 .4678903E-05 .2833640E-04 -.6465519E-04 -.8689381E-04 .7296172E-04 .1210706E-04 .8566833E-04 .2067967E-03 .3410982E-03 .4527225E-03 .4872474E-03 .4643345E-03 .4422057E-03 .4129112E-03 .3829180E-03 .3780384E-03 .3995229E-03 .4325189E-03 .4715937E-03 .5721359E-03 .7172794E-03 .8375864E-03 .9064132E-03 .9303048E-03 .8492444E-03 .6122706E-03 .2602388E-03 .5337829E-04 -.3082092E-03 .4980709E-03 .5981078E-03 .6193480E-03 .4957118E-03 -.2834890E-03 .1072630E-04 .4309251E-03 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APPENDIX I:**PEER Ground Motion Data Base****RSN454_MORGAN_GIL337.AT2**

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 -.1822897E-01 -.1151862E-02 .3139245E-02 .4320946E-02 .9194479E-02
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 .3364579E-02 .2563524E-02 .1458875E-02 .7438318E-03 .6755445E-03
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 .1247214E-01 .3233826E-02 .7832390E-02 .1950690E-01 .295671E-01
 .3681613E-01 .4009524E-01 .3979842E-01 .3645640E-01 .3071803E-01
 .2352556E-01 .1588706E-01 .8715613E-02 .2695112E-02 .1720591E-02
 .4641173E-02 .6027574E-02 .5618362E-02 .3725959E-02 .1206954E-02
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 .2957973E-01 .3355001E-01 .3822404E-01 .4292526E-01 .4610961E-01
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APPENDIX II:**PEER Ground Motion Data Base****RSN1633_MANJIL_ABBAR--L.AT2**

-2833101E+00 -2022618E+00 -1517949E+00 -1397840E+00 -7225265E-01
 -2191203E-01 .2153031E-01 .1727688E+00 .1965910E+00 .8875494E-01
 .3094025E-01 .1989580E-01 .4508748E-01 .1374721E+00 .2467463E+00
 .3055493E+00 .1970094E+00 .3485844E-01 .4889407E-01 .1101575E+00
 -.3419411E+00 .4384356E+00 .4096841E+00 .3569527E+00 .1285304E+00
 -.3391772E-01 .1503853E-01 .6393292E-01 .1694953E+00 .2885540E+00
 .3205704E+00 .3176403E+00 .3089821E+00 .1936700E+00 .3640686E-01
 -.2532655E+00 .4042270E+00 .3676818E+00 .6092999E-01 .2104664E+00
 .2038656E+00 .1646652E+00 .4562823E-01 .7995949E-01 .6720194E-01
 .1307683E+00 .8389002E-01 .9450901E-01 .1477821E+00 .1172669E+00
 .1769342E+00 .1979327E+00 .1686279E+00 .8820137E-01 .8821092E-01
 -.2569743E+00 .3740821E+00 .2749871E+00 .7720250E-01 .8764973E-01
 .4140850E-01 .4844055E-01 .1107560E+00 .1043417E-01 .4269893E-01
 .8740689E-01 .1033302E-01 .6037564E-01 .1247810E+00 .1570051E+00
 .2152305E-02 .8153688E-02 .9773280E-01 .1522118E+00 .1114915E+00
 -.8677784E-01 .3317979E-01 .6288075E-01 .4429955E-01 .4197988E-01
 -.1418696E-01 .4647740E-01 .8959954E-02 .4976402E-01 .1682096E-01
 .1429692E-01 .2602500E-01 .3761819E-01 .4382666E-01 .2485514E-01
 .3947924E-01 .1426115E+00 .1561810E+00 .6984973E-01 .5256157E-01
 -.3013273E-01 .1241720E+00 .8637677E-01 .5407047E-01 .7787811E-01
 .2719623E-01 .1029491E+00 .7232696E-01 .4265431E-01 .2419990E-02
 .2290623E-01 .1372175E+00 .1229647E+00 .2604950E-01 .2033516E-01
 .5307415E-01 .7449912E-01 .2549660E-01 .1295917E+00 .6704105E-01
 .4415553E-01 .3099570E-01 .9326561E-02 .3284459E-01 .3482190E-01
 .1032161E+00 .8840266E-01 .4346235E-01 .1391108E+00 .1137732E+00
 .9938141E-01 .9214638E-01 .7439986E-01 .3278459E-01 .4930123E-01
 .5260035E-01 .8405655E-01 .6096484E-01 .1460502E+00 .1220124E-01
 .5328443E-01 .1411257E-02 .6207457E-02 .2020574E-01 .5048287E-01
 .7941961E-01 .2511851E-01 .5194845E-01 .1293743E+00 .1369378E+00
 .1012070E+00 .1915729E+00 .2704553E+00 .2746854E-01 .17394118E-01
 .9855430E-01 .5884625E-01 .7647532E-01 .8461500E-01 .7598985E-01
 .2399491E+00 .2404007E+00 .6718076E-01 .1166696E+00 .1400374E+00
 .1138401E-01 .1876417E+00 .3676344E-01 .2788189E+00 .1164410E+00
 .6480600E-01 .4098352E-01 .5757399E-02 .1583555E+00 .3151813E+00
 .2745606E+00 .2896234E+00 .2032737E+00 .1146107E+00 .5687418E-02
 .2412573E-01 .2025259E-01 .5660447E-01 .1205056E+00 .2655956E+00
 .3119874E+00 .2567038E+00 .8872238E-01 .2193059E+00 .1739568E+00
 .3279278E+00 .3390734E+00 .7709963E-01 .2340215E+00 .2391532E+00
 .3666418E-01 .7597812E-01 .1898858E+00 .1424847E+00 .5878989E-01
 .1283905E+00 .4708743E-02 .2890806E+00 .1353570E-01 .2128425E+00
 .2666516E+00 .1272666E+00 .2139276E+00 .1915891E+00 .1221578E+00
 .2137425E+00 .3488561E+00 .1327266E+00 .3371555E-01 .3047427E+00
 .1130715E+00 .4363288E+00 .3202360E+00 .7297481E-01 .1663819E+00
 .2043369E+00 .2125411E-01 .8125758E-02 .4264371E-01 .1513757E+00
 .1455978E+00 .1362603E+00 .2682441E-01 .2200340E+00 .2120701E+00
 .2463495E+00 .1149572E+00 .2032961E+00 .7141812E-01 .1391770E+00
 .4930915E-01 .4406646E-01 .1586623E+00 .1043840E+00 .1247295E+00
 .3326858E-01 .1863046E+00 .2546036E+00 .3468795E+00 .2908039E+00
 .6652101E-01 .7708748E-01 .8569343E-01 .8828477E-01 .2001242E+00
 .4471648E-01 .4249380E-01 .1396951E+00 .7394991E-01 .1523070E+00
 .1173121E-01 .1413115E+00 .1677753E+00 .7750532E-01 .3082632E+00
 .5145641E+00 .4852056E+00 .1062753E+00 .1268331E+00 .2168071E+00
 .3376479E+00 .1892256E+00 .2605321E+00 .3594968E+00 .2754992E+00
 .4276334E+00 .2893615E+00 .1564001E+00 .2787396E+00 .3093995E+00
 .2929415E+00 .7913653E-01 .1693827E+00 .3185418E-01 .6292118E-01
 .6901570E-01 .3254914E+00 .4180183E+00 .3551806E+00 .2964243E+00
 .2786125E+00 .8216169E-01 .2630361E+00 .1141828E-01 .1216715E+00
 .2817859E+00 .7011975E-01 .2455525E-01 .1796226E+00 .1698366E+00
 .2009186E-01 .2174499E+00 .2255771E+00 .1109266E+00 .1515569E+00
 .1334927E+00 .1787936E+00 .1172351E+00 .8417712E-01 .1052598E+00
 .9068310E-01 .1085774E+00 .1028883E+00 .7637065E-01 .7835246E-01
 .1454819E+00 .8353750E-01 .5113944E-01 .5002443E-03 .9368145E-01
 .1077335E+00 .7545400E-01 .7201518E-01 .1273390E+00 .2144963E+00
 .2210957E+00 .2430246E+00 .1173689E+00 .1780882E+00 .2437670E+00

APPENDIX I:**PEER Ground Motion Data Base****RSN454_MORGAN_GIL337.AT2**

.1489746E-01 .9720374E-02 .5092390E-02 .3499935E-03 .4556741E-02
 -.9680059E-02 -.1540586E-01 .2248652E-01 .3074179E-01 .3973322E-01
 -.4886946E-01 .5724180E-01 .6381594E-01 .6788051E-01 .6920425E-01
 -.6800116E-01 .6444003E-01 .5804576E-01 .4889914E-01 .3795440E-01
 -.2665668E-01 .1623631E-01 .7819260E-02 .1750119E-02 .2264038E-02
 .5157081E-02 .8153538E-02 .1246923E-01 .1856924E-01 .2612751E-01
 .3393523E-01 .4042602E-01 .4468596E-01 .4583467E-01 .4333538E-01
 .3777785E-01 .2978261E-01 .2016726E-01 .9832265E-02 .4231183E-03
 -.9696577E-02 .1698112E-01 .2187655E-01 .2451724E-01 .2456441E-01
 -.2212270E-01 .1784224E-01 .1271201E-01 .7476224E-02 .2378402E-02
 .2292519E-02 .6275419E-02 .9967592E-02 .1337392E-01 .1578106E-01
 .1662687E-01 .1618647E-01 .1509638E-01 .1380727E-01 .1322182E-01
 .1341152E-01 .1355263E-01 .1279392E-01 .1016191E-01 .5842929E-02
 .1233927E-02 .2423577E-02 .4624110E-02 .5557404E-02 .5794732E-02
 -.5369056E-02 .4014008E-02 .9779942E-03 .3923870E-02 .9516564E-02
 .1450664E-01 .1798926E-01 .1925302E-01 .1841348E-01 .1621111E-01
 .1356946E-01 .1085105E-01 .7813205E-02 .4894075E-02 .3162055E-02
 .3123575E-02 .4747080E-02 .6939272E-02 .8280652E-02 .7835319E-02
 .5028185E-02 .2976133E-03 .5418406E-02 .1153247E-01 .1779071E-01
 -.2412134E-01 .2995210E-01 .3434539E-01 .3678776E-01 .3687265E-01
 -.3481515E-01 .3116871E-01 .2651753E-01 .2160757E-01 .1671211E-01
 -.1146653E-01 .5534051E-02 .1482148E-02 .9377325E-02 .1739977E-01
 .2418828E-01 .2831337E-01 .2903885E-01 .2646692E-01 .2133763E-01
 .1427232E-01 .5538088E-02 .4425434E-02 .1489936E-01 .2550815E-01
 -.3545918E-01 .4429352E-01 .5171073E-01 .5696568E-01 .5966265E-01
 -.5958343E-01 .5641559E-01 .5063785E-01 .4326230E-01 .3535192E-01
 -.2771292E-01 .2094448E-01 .1514580E-01 .9841592E-02 .4336806E-02
 .1887544E-02 .8934472E-02 .1665178E-01 .2425928E-01 .3064779E-01
 .3547308E-01 .3913809E-01 .4216289E-01 .4500158E-01 .4746660E-01
 .4890939E-01 .4891160E-01 .4730507E-01 .4500518E-01 .4313362E-01
 .4241865E-01 .4256683E-01 .4268163E-01 .4139644E-01 .3717918E-01
 .2904273E-01 .1679912E-01 .1825046E-02 .1358298E-01 .2789135E-01
 -.3948239E-01 .4750243E-01 .5199208E-01 .5309432E-01 .5129485E-01
 -.4742314E-01 .4145453E-01 .3329664E-01 .2370838E-01 .1339123E-01
 -.2850918E-02 .7277556E-02 .1618460E-01 .2386917E-01 .3063999E-01
 .3641412E-01 .4125706E-01 .4462245E-01 .4555109E-01 .4322025E-01
 .3777520E-01 .3054856E-01 .2340481E-01 .1765535E-01 .1401578E-01
 .1258863E-01 .1205708E-01 .1071905E-01 .7708976E-02 .2537865E-02
 -.4591097E-02 .1241342E-01 .1973711E-01 .2630334E-01 .3213413E-01
 -.3704646E-01 .4088336E-01 .4323561E-01 .4331094E-01 .3998685E-01
 -.3288381E-01 .2280285E-01 .1144198E-01 .3325743E-03 .9780855E-02
 .1840444E-01 .2448580E-01 .2682036E-01 .2485258E-01 .1847352E-01
 .8771153E-02 .2092048E-02 .1187043E-01 .1919526E-01 .2341986E-01
 -.2426598E-01 .2206456E-01 .1749916E-01 .1131764E-01 .4458513E-02
 .1949138E-02 .6696217E-02 .9291466E-02 .1000604E-01 .9424715E-02
 .8550144E-02 .8211624E-02 .8922633E-02 .1032583E-01 .1131955E-01
 .1065658E-01 .7799977E-02 .3033368E-02 .2736048E-02 .7892268E-02
 -.1121344E-01 .1258233E-01 .1299998E-01 .1371240E-01 .1541357E-01
 -.1831175E-01 .2193011E-01 .2587833E-01 .2974404E-01 .3273323E-01
 -.3456496E-01 .3521929E-01 .3493035E-01 .3337685E-01 .2995297E-01
 -.2471179E-01 .1746893E-01 .8007992E-02 .2679110E-02 .1328190E-01
 .2269222E-01 .3013368E-01 .3590930E-01 .4073028E-01 .4508395E-01
 .4878093E-01 .5133643E-01 .5209759E-01 .5042930E-01 .4578930E-01
 .3845582E-01 .2919670E-01 .1908564E-01 .9817011E-02 .2548925E-02
 -.2181196E-02 .4610896E-02 .5345476E-02 .5250207E-02 .5202501E-02
 -.5803980E-02 .7084765E-02 .8565304E-02 .9184876E-02 .8132516E-02
 -.5683238E-02 .2743632E-02 .2915653E-03 .1175361E-02 .1123872E-02
 -.1340208E-04 .1985378E-02 .4606999E-02 .7708314E-02 .1089410E-01
 -.1372997E-01 .1558643E-01 .1630542E-01 .1661211E-01 .1666824E-01
 -.1644782E-01 .1554405E-01 .1362877E-01 .1067974E-01 .7333290E-02
 -.4643016E-02 .3278280E-02 .3060572E-02 .3306367E-02 .3474182E-02
 -.3395323E-02 .3095248E-02 .2172062E-02 .9047009E-03 .5290165E-03
 .1723792E-02 .2351309E-02 .3168581E-02 .4956918E-02 .7952341E-02
 .1178831E-01 .1556419E-01 .1863218E-01 .2035166E-01 .1999128E-01

APPENDIX II:**PEER Ground Motion Data Base****RSN1633_MANJIL_ABBAR--L.AT2**

.2234653E+00 .5195897E-01 .1887896E+00 .2857136E+00 .1038943E+00
 .8884539E-01 .1752351E+00 .1716058E+00 .1801826E+00 .1553549E+00
 .8436803E-01 .6517919E-01 .7082959E-01 .1660560E+00 .7422457E-01
 -.5751445E-01 .1477752E+00 .1622940E+00 .1147508E-01 .2796659E-01
 -.7451751E-01 .9979207E-01 .7786800E-01 .4056945E-01 .1413238E+00
 .1114428E+00 .1607920E+00 .2323719E+00 .2066085E+00 .2705474E+00
 -.2400748E+00 .7589573E-01 .8666877E-01 .1225556E+00 .7025178E-01
 .1624217E+00 .1937246E+00 .1550557E+00 .1771977E+00 .1819690E-01
 -.1084282E+00 .3139493E-01 .1282767E+00 .1651847E+00 .3435397E-01
 -.5940705E-01 .1052572E-01 .2132924E-01 .3336723E-01 .3391245E-01
 -.1290130E+00 .1669867E+00 .1062643E+00 .8968023E-01 .4879507E-01
 .1346395E+00 .8633921E-01 .8939762E-01 .6631774E-02 .1650284E+00
 -.1571803E+00 .1003432E+00 .1363435E+00 .5212927E-01 .2945076E+00
 .2854873E+00 .2269353E+00 .3640133E+00 .1454998E+00 .3029553E+00
 -.7429746E-01 .4900896E-01 .5353160E-01 .5047787E-01 .7497191E-01
 -.1574488E+00 .6719626E-02 .5585395E-01 .1009667E+00 .1813022E+00
 -.1865608E-01 .1197331E+00 .2855686E-02 .1226956E-01 .4095131E-01
 -.8178093E-01 .1352896E+00 .5974335E-01 .2998092E-01 .5233017E-01
 .6769254E-01 .1019156E+00 .1132606E+00 .1212324E+00 .4351926E-03
 -.3678126E-01 .2970702E-01 .2011502E-01 .9610233E-01 .2313146E+00
 .8092654E-01 .2790497E-01 .2713482E-01 .9191779E-01 .1003066E+00
 -.3795591E-01 .2189467E-01 .3369566E-01 .2897392E-01 .2851027E-01
 .2263619E-01 .8156104E-01 .1077181E+00 .7960906E-01 .8058039E-01
 .3464713E-01 .1145035E-01 .3134086E-01 .6199914E-01 .9108734E-01
 -.7128884E-01 .4271680E-02 .4620221E-01 .2177704E-01 .7762638E-01
 -.1830839E+00 .2566211E+00 .2722584E+00 .1808491E+00 .8238237E-02
 .2417899E+00 .2770448E+00 .2036441E+00 .2620713E+00 .2016165E+00
 -.6197499E-01 .1149964E+00 .2506198E-01 .9081141E-01 .5901498E-01
 .2600870E-01 .3390398E-01 .6342255E-02 .5190732E-01 .1955256E-01
 .5404227E-01 .1017721E+00 .2545130E-01 .1829884E+00 .4412259E-02
 .7521318E-01 .2069519E+00 .9243665E-01 .2827281E-01 .9487676E-01
 -.9521542E-01 .1299970E-01 .1293485E+00 .1685363E+00 .1441246E+00
 -.1390513E+00 .8188059E-01 .1324412E+00 .2154592E-01 .4290116E-01
 -.4438576E-01 .1090444E-01 .7721610E-01 .1131721E+00 .6982874E-01
 -.2216066E-01 .7786680E-01 .3376880E-01 .6622448E-01 .7306387E-01
 .5879147E-01 .9932263E-02 .6153960E-01 .1560184E-01 .2779768E-01
 -.6404089E-01 .7261475E-01 .3345627E-01 .5089471E-02 .3368922E-01
 -.4194138E-01 .2520839E-02 .2212844E-01 .8696895E-02 .9489685E-02
 .9632693E-02 .3383135E-01 .3538471E-01 .3305797E-02 .4188315E-02
 -.8468781E-02 .2329842E-01 .9250327E-02 .1160588E-01 .3206419E-01
 .8855190E-03 .7420373E-02 .4996114E-01 .3030618E-01 .1750146E-01
 -.2735115E-01 .4000422E-02 .7771991E-02 .1920683E-02 .4368539E-02
 -.1953445E-01 .4927400E-01 .4073610E-01 .2099586E-01 .2517171E-01
 .1529349E-01 .2871698E-01 .3217513E-01 .5089471E-02 .1126884E-01
 .3177521E-01 .4602929E-01 .1505994E-01 .1694809E-01 .2243846E-01
 .3524899E-01 .9268973E-01 .1014530E+00 .9366165E-01 .7544237E-01
 -.1338483E-01 .6054687E-01 .5687875E-01 .3883869E-01 .2349581E-01
 -.5428661E-01 .5311421E-01 .5784641E-01 .6230449E-01 .2978919E-01
 .1153560E-01 .2213287E-01 .3389021E-01 .2200496E-01 .7436211E-02
 -.3845571E-02 .2045073E-01 .1489951E-01 .1505333E-01 .1325347E-01
 -.1075304E-01 .2863430E-01 .1746461E-01 .4035562E-04 .2376560E-02
 .3663766E-01 .3430852E-01 .2738477E-02 .2977542E-01 .3867641E-01
 .2269499E-01 .5880425E-01 .7086181E-01 .2710785E-01 .2405772E-01
 -.4383329E-01 .2355034E-01 .1592822E-01 .2833824E-01 .1508830E-01
 .2475935E-01 .4951871E-01 .1560484E-01 .3861059E-01 .4870306E-01
 -.1853682E-01 .8375615E-02 .2598012E-02 .3633855E-01 .3489699E-01
 -.7552654E-02 .1453277E-01 .1161395E-01 .3040488E-01 .3159348E-02
 .2828180E-02 .9915944E-02 .5880190E-02 .1429226E-01 .1257420E-01
 .3665105E-02 .3707031E-01 .6485596E-01 .7019695E-01 .6209116E-01
 .6497826E-01 .4208024E-01 .1003321E-01 .2436247E-01 .4452404E-01
 .4645479E-01 .1359755E-01 .4637730E-02 .1431034E-01 .3461873E-01
 .4117094E-01 .2238081E-01 .1117060E-01 .3012335E-01 .1379586E-01
 -.5320944E-02 .1576724E-01 .3269448E-01 .1585247E-01 .2808316E-01
 -.4242819E-01 .4430869E-01 .6057650E-01 .5570094E-01 .3859596E-01

APPENDIX I: PEER Ground Motion Data Base RSN454_MORGAN_GIL337.AT2

.1711756E-01 .1204014E-01 .6031453E-02 .3485173E-03 .4039141E-02
 -.6955157E-02 -.8201193E-02 -.8137363E-02 -.7053713E-02 -.5179244E-02
 -.3258085E-02 -.2461960E-02 -.3548001E-02 -.6409180E-02 -.1026417E-01
 -.1389806E-01 -.1596348E-01 -.1563137E-01 -.1328024E-01 -.9500824E-02
 -.4753668E-02 .7415868E-03 .6816767E-02 .1282339E-01 .1762850E-01
 .2030862E-01 .2135969E-01 .2174285E-01 .2196637E-01 .2191065E-01
 .2158572E-01 .2059119E-01 .1851361E-01 .1586013E-01 .1370422E-01
 .1197948E-01 .9992201E-02 .6946107E-02 .2075160E-02 -.4636816E-02
 -.1192623E-01 -.1809487E-01 -.2262826E-01 -.2611568E-01 -.2912783E-01
 -.3183501E-01 -.3355103E-01 -.3318501E-01 -.3008709E-01 -.2442112E-01
 -.1669827E-01 -.7873014E-02 .7875856E-03 .8234546E-02 .1381376E-01
 .1749069E-01 .1965685E-01 .2054500E-01 .2006485E-01 .1864002E-01
 .1656700E-01 .1371191E-01 .9948303E-02 .4860940E-02 -.1870338E-02
 -.1014390E-01 -.1927521E-01 -.2813209E-01 -.3562035E-01 -.4058484E-01
 -.4198004E-01 -.3919339E-01 -.3269411E-01 -.2319316E-01 -.1162725E-01
 .8250332E-03 .1344265E-01 .2451879E-01 .3246850E-01 .3653495E-01
 .3667645E-01 .3353225E-01 .2824004E-01 .2211832E-01 .1627732E-01
 .1113899E-01 .6972710E-02 .4534949E-02 .4226854E-02 .5831906E-02
 .8720836E-02 .1191604E-01 .1476932E-01 .1724645E-01 .1962283E-01
 .2207202E-01 .2430450E-01 .2582395E-01 .2572697E-01 .2372660E-01
 .2028818E-01 .1663857E-01 .1374144E-01 .1207125E-01 .1168425E-01
 .1264527E-01 .1494909E-01 .1802008E-01 .2136394E-01 .2417732E-01
 .2550987E-01 .2472739E-01 .2149939E-01 .1640790E-01 .1000758E-01
 .2631452E-02 -.5096445E-02 -.1300941E-01 -.2056565E-01 -.2741053E-01
 -.3302135E-01 -.3667198E-01 -.3833158E-01 -.3828216E-01 -.3724918E-01
 -.3596011E-01 -.3463880E-01 -.3293091E-01 -.3064446E-01 -.2796313E-01
 -.2501167E-01 -.2162766E-01 -.1768681E-01 -.1333574E-01 -.9528792E-02
 -.6988929E-02 -.5382158E-02 -.3694268E-02 -.1361064E-02 .1505209E-02
 .4495493E-02 .7166159E-02 .9161187E-02 .1070227E-01 .1221487E-01
 .1337433E-01 .1391335E-01 .1398474E-01 .1390490E-01 .1359808E-01
 .1255143E-01 .1083571E-01 .8583953E-02 .6113238E-02 .3774435E-02
 .1854873E-02 .4327453E-03 -.8305361E-03 -.2077670E-02 -.3449798E-02
 -.4445009E-02 -.4620481E-02 -.3856802E-02 -.2098163E-02 .8368638E-03
 .4806354E-02 .9377475E-02 .1381897E-01 .1723555E-01 .1863260E-01
 .1746501E-01 .1355298E-01 .7251015E-02 -.8503565E-03 -.9650233E-02
 -.1767852E-01 -.2366815E-01 -.2702183E-01 -.2789853E-01 -.2714535E-01
 -.2526755E-01 -.2235196E-01 -.1880084E-01 -.1450777E-01 -.9513325E-02
 -.4151793E-02 .1080907E-02 .6140817E-02 .1146465E-01 .1657137E-01
 .2023506E-01 .2181359E-01 .2191304E-01 .2159490E-01 .2132119E-01
 .2086273E-01 .2020429E-01 .1950061E-01 .1858467E-01 .1752428E-01
 .1589410E-01 .1252968E-01 .6806147E-02 -.7957274E-03 -.9650771E-02
 -.1915393E-01 -.2831264E-01 -.3577399E-01 -.4022506E-01 -.4164743E-01
 -.4088224E-01 -.3833411E-01 -.3428432E-01 -.2913666E-01 -.2376154E-01
 -.1928907E-01 -.1612262E-01 -.1452131E-01 -.1459422E-01 -.1594008E-01
 -.1753592E-01 -.1834704E-01 -.1801922E-01 -.1673808E-01 -.1431030E-01
 -.1037721E-01 -.5093443E-02 .8711377E-03 .6942120E-02 .1279369E-01
 .1776915E-01 .2167639E-01 .2456358E-01 .2573498E-01 .2483765E-01
 .2300371E-01 .2121156E-01 .1942259E-01 .1783568E-01 .1637813E-01
 .1438554E-01 .1170686E-01 .8722798E-02 .5748507E-02 .2306153E-02
 -.1812500E-02 -.6514054E-02 -.1165813E-01 -.1693575E-01 -.2155924E-01
 -.2530562E-01 -.2807732E-01 -.2987431E-01 -.3043874E-01 -.2944895E-01
 -.2692176E-01 -.2366249E-01 -.2050365E-01 -.1715798E-01 -.1358334E-01
 -.9574298E-02 -.4815477E-02 .4543519E-03 .5238787E-02 .9144373E-02
 .1228349E-01 .1458535E-01 .1587569E-01 .1631722E-01 .1635640E-01
 .1644066E-01 .1733816E-01 .1942244E-01 .2258076E-01 .2627993E-01
 .2900391E-01 .2928129E-01 .2718093E-01 .2358762E-01 .1881119E-01
 .1311578E-01 .7581341E-02 .3285314E-02 .2784876E-03 -.2060554E-02
 -.3835469E-02 -.5452050E-02 .6856066E-02 -.7670810E-02 .7904010E-02
 -.7840258E-02 -.7773372E-02 -.7715676E-02 -.7007358E-02 -.5161399E-02
 -.2062555E-02 .2535561E-02 .8136502E-02 .1386052E-01 .1913482E-01
 .2306537E-01 .2518611E-01 .2508626E-01 .2311539E-01 .2035478E-01
 .1761539E-01 .1505357E-01 .1239909E-01 .9850222E-02 .7583947E-02
 .5882795E-02 .4262215E-02 .2061988E-02 -.1166369E-02 -.5579639E-02
 -.1081159E-01 -.1618535E-01 -.2076260E-01 -.2340955E-01 -.2391080E-01

APPENDIX II: PEER Ground Motion Data Base RSN1633_MANJIL_ABBAR--L.AT2

-.1610947E-01 .2597335E-01 .3680423E-01 .3676936E-01 .2502172E-01
 .9607953E-02 -.8540590E-02 -.2557073E-01 -.5074787E-01 -.6090285E-01
 -.4040206E-01 -.5102912E-01 -.2657321E-01 -.1454611E-01 -.4252617E-01
 -.6634477E-01 -.6240537E-01 -.1592039E-01 .3395634E-01 .2743934E-01
 -.1057477E-01 -.5743982E-02 -.1425053E-01 .1988470E-02 .2727514E-01
 .1435494E-01 .1036278E-01 -.1147643E-01 .3765715E-01 -.3545443E-01
 -.2505142E-01 .2021378E-02 .1505524E-01 .1633039E-01 .4025011E-01
 .6716238E-01 .6047770E-01 .3566037E-01 .1431945E-01 -.1364811E-01
 -.2212615E-01 -.1607034E-01 .1845900E-01 -.2745726E-02 -.1065041E-01
 -.8630855E-02 .9248541E-02 .1302192E-01 .1419195E-01 -.1308578E-01
 .1118846E-01 .6135041E-01 .2620667E-01 .4150923E-01 -.3084090E-01
 .3179138E-01 .3963527E-01 .4513131E-02 .1839490E-01 -.1750457E-01
 -.1483652E-01 .3520872E-01 .4314997E-01 .1164608E-01 .3218154E-01
 .1166497E-01 -.8903808E-02 .6351271E-02 -.8164964E-02 -.1666985E-01
 .6640866E-02 .2396306E-01 .4408447E-01 .3198213E-01 -.1453798E-01
 -.3460416E-01 -.5735990E-01 .5934894E-01 .2095051E-01 .1380317E-01
 .7855903E-02 -.4556992E-01 .7023972E-01 .5400849E-01 .5967613E-01
 -.3585655E-01 .6182627E-02 .8362005E-02 .2488844E-01 -.1636337E-02
 -.9313955E-02 -.4035828E-01 .1078451E-01 .4285986E-02 .2943544E-01
 -.4039773E-01 -.2116390E-01 .5827729E-03 .1414727E-01 .3351238E-01
 .2842647E-02 .1360686E-01 .3733380E-02 .1030757E-01 .2741380E-02
 .3257509E-02 .1958592E-01 .3869624E-01 .2473540E-01 .3028413E-02
 .4214546E-01 .6354597E-01 .5181280E-01 .2146042E-01 .2121753E-01
 .3057554E-01 .1276060E-02 .4358336E-02 .4058850E-01 .2959091E-01
 .3195641E-02 .9084534E-02 .4870824E-02 .5420475E-02 .9532336E-02
 .5382262E-02 .2645949E-01 .5978513E-01 .5206497E-01 .1597972E-01
 .5767457E-02 .2892321E-01 .1957273E-01 .1893777E-01 .1476941E-01
 .2033526E-01 .1740317E-01 .2078895E-01 .2143930E-01 .3895179E-03
 .9992311E-02 .3833634E-02 .2679058E-01 .1230944E-01 .2322455E-02
 .1571135E-01 .1039677E-01 .2386015E-02 .1781281E-01 .1336319E-01
 .2331109E-01 .2991021E-01 .1211090E-01 .3812897E-02 .1006895E-01
 .2828897E-01 .3591007E-01 .7404102E-02 .4614925E-01 .5588809E-01
 .2983719E-03 .1855036E-01 .1807286E-01 .1467932E-02 .6017580E-01
 .4631035E-01 .1137132E-01 .1428681E-01 .3555941E-01 .4263071E-01
 .5619669E-01 .1752994E-02 .4752168E-01 .2896981E-01 .6853822E-02
 .5163662E-01 .3625509E-01 .3609568E-01 .4951290E-01 .1468949E-01
 .1359376E-01 .1433564E-01 .1178784E-01 .7456588E-03 .7114138E-02
 .1488712E-01 .1099000E-01 .7390344E-02 .1399591E-01 .4342765E-02
 .2959766E-02 .5068069E-02 .1370717E-01 .7907528E-02 .2116332E-02
 .1694294E-01 .1880816E-01 .3043878E-01 .4965113E-01 .1443699E-01
 .3600054E-01 .3773982E-01 .8490649E-02 .2080385E-01 .3198173E-01
 .2624802E-01 .6803828E-02 .1519037E-01 .1970350E-01 .1430830E-01
 .2384281E-02 .1980458E-01 .3880342E-01 .2446212E-01 .1141942E-01
 .6246566E-02 .1380622E-01 .6220251E-02 .1042732E-01 .1206843E-01
 .4083830E-02 .3642593E-01 .5970606E-01 .3379561E-01 .9541894E-02
 .3286239E-01 .1891792E-01 .3763427E-02 .1131775E-01 .1135373E-01
 .1368183E-01 .1269099E-01 .2931899E-02 .2898597E-02 .1254915E-02
 .2445326E-01 .6947143E-02 .1952212E-01 .3177920E-01 .1930273E-01
 .3254876E-02 .2243666E-01 .1889456E-01 .2459023E-02 .9080255E-02
 .6596783E-02 .2485552E-01 .2169208E-01 .4370266E-02 .4941578E-02
 .1531261E-01 .1713237E-01 .1692399E-01 .4832563E-01 .3340303E-01
 .1584417E-01 .7261388E-01 .8806067E-01 .3366052E-01 .3445151E-02
 .7639193E-02 .1364196E-02 .3349932E-01 .2400628E-01 .1071529E-01
 .1117113E-01 .1668956E-01 .3734037E-01 .6876288E-01 .7804652E-01
 .3603222E-01 .1433171E-01 .4876420E-01 .4152538E-01 .4644900E-02
 .1618486E-01 .2535920E-01 .3260534E-01 .1555633E-01 .1989199E-01
 .2697573E-01 .2553049E-01 .8364451E-02 .1699390E-01 .8051409E-02
 .5310016E-02 .1717545E-01 .3118898E-01 .2341921E-01 .1800648E-02
 .9239753E-02 .3919182E-01 .5071226E-01 .1862419E-01 .2089124E-01
 .4545234E-01 .2874965E-01 .1730764E-01 .5696526E-03 .4338487E-01
 .2773575E-01 .3274423E-01 .4224171E-01 .1586004E-01 .8074750E-02
 .1374952E-01 .2839560E-01 .1876803E-01 .2135419E-01 .8818396E-02
 .1284686E-01 .2092308E-01 .1161528E-01 .3401433E-01 .2971191E-01
 .1859769E-01 .4890377E-01 .3152007E-01 .5371293E-01 .5552031E-01

APPENDIX I: PEER Ground Motion Data Base RSN454_MORGAN_GIL337.AT2	APPENDIX II: PEER Ground Motion Data Base RSN1633_MANJIL_ABBAR--L.AT2
- .2244776E-01 -.1928988E-01 -.1544236E-01 -.1145384E-01 -.7871212E-02 -.4930642E-02 -.2570008E-02 -.1068460E-02 -.3660948E-03 .1553651E-03 .7687791E-03 .1565657E-02 .2266584E-02 .2702937E-02 .3152095E-02 .3493465E-02 .3318324E-02 .2657441E-02 .1297871E-02 .7501890E-03 -.2860052E-02 -.4472787E-02 -.5086924E-02 -.4186400E-02 -.1782680E-02 .1850650E-02 .5469557E-02 .7330803E-02 .5974583E-02 .8273618E-03 -.7359078E-02 -.1705510E-01 -.2625069E-01 -.3328723E-01 -.3768642E-01 -.4020138E-01 -.4144054E-01 -.4170421E-01 -.4151069E-01 .4092424E-01 -.3937190E-01 -.3603579E-01 -.3062291E-01 -.2359669E-01 -.1585543E-01 -.7387137E-02 .1651380E-02 .1035034E-01 .1729114E-01 .2092496E-01 .2071798E-01 .1739925E-01 .1291617E-01 .9213799E-02 .7211587E-02 .6839769E-02 .7754805E-02 .9945046E-02 .1309838E-01 .1646027E-01 .1984096E-01 .2330865E-01 .2661596E-01 .2948757E-01 .3187992E-01 .3441449E-01 .3730473E-01 .4030986E-01 .4310627E-01 .4511985E-01 .4557893E-01 .4423479E-01 .4160314E-01 .3814718E-01 .3367141E-01 .2739305E-01 .1957511E-01 .1118234E-01 .3014020E-02 -.4234317E-02 -.9871731E-02 -.1358964E-01 -.1538021E-01 -.1586906E-01 -.1607962E-01 -.1710668E-01 -.1913516E-01 -.2212677E-01 -.2570067E-01 -.2904499E-01 -.3201618E-01 -.3414728E-01 -.3507544E-01 -.3508990E-01 -.3423499E-01 -.3313108E-01 -.3250730E-01 -.3237914E-01 -.3171627E-01 -.2914893E-01 -.2413187E-01 -.1707654E-01 -.8673179E-02 .3146191E-03 .8679244E-02 .1542811E-01 .2013342E-01 .2327030E-01 .2583037E-01 .2846130E-01 .3100660E-01 .3304434E-01 .3408230E-01 .3315853E-01 .2979019E-01 .2462925E-01 .1856046E-01 .1224453E-01 .6287214E-02 .1245373E-02 -.2463271E-02 -.4106030E-02 -.3120482E-02 -.2597387E-03 .3638167E-02 .7787446E-02 .1098858E-01 .1229507E-01 .1050616E-01 .5792666E-02 -.7927429E-03 -.8006062E-02 -.1514790E-01 -.2158736E-01 -.2645791E-01 -.2937490E-01 -.3056480E-01 -.3045384E-01 -.2959005E-01 -.2845738E-01 -.2730911E-01 -.2621981E-01 -.2535241E-01 -.2416750E-01 -.2170571E-01 -.1688337E-01 -.9365379E-02 .2017124E-03 .1033430E-01 .1923113E-01 .2557457E-01 .2868215E-01 .2893833E-01 .2740152E-01 .2507644E-01 .2227754E-01 .1859421E-01 .1328982E-01 .6945082E-02 .1096944E-02 -.3131086E-02 -.5225867E-02 .5601407E-02 -.4992214E-02 .3927674E-02 -.2348163E-02 .4542141E-03 .4430255E-02 .8728400E-02 .1173559E-01 .1207967E-01 .1001472E-01 .6944068E-02 .4429276E-02 .3690870E-02 .4705929E-02 .6515740E-02 .8499911E-02 .1015465E-01 .1065166E-01 .9488036E-02 .7016538E-02 .3757284E-02 .8309348E-04 .3728293E-02 -.7517984E-02 -.1090681E-01 .1310913E-01 .1391083E-01 .1390617E-01 -.1369491E-01 .1303327E-01 .1214560E-01 .1158211E-01 .1145900E-01 -.1156562E-01 .1158289E-01 .1069171E-01 .7878585E-02 .2875331E-02 .3243880E-02 .9134267E-02 .1357710E-01 .1536728E-01 .1408842E-01 .1093329E-01 .7386536E-02 .4999725E-02 .4403832E-02 .5222047E-02 .6711932E-02 .8518849E-02 .1053210E-01 .1212344E-01 .1315753E-01 .1405571E-01 .1483064E-01 .1534272E-01 .1586807E-01 .1645769E-01 .1718572E-01 .1786852E-01 .1823388E-01 .1824879E-01 .1781869E-01 .1718701E-01 .1648362E-01 .1591183E-01 .1558102E-01 .1486232E-01 .1375375E-01 .1220849E-01 .1005548E-01 .7441808E-02 .4344022E-02 .7662474E-03 .2686862E-02 .4952786E-02 .5665527E-02 .5110600E-02 -.3943394E-02 .3141504E-02 .4111722E-02 .7558555E-02 .1281956E-01 -.1895034E-01 .2528646E-01 .3111422E-01 .3528622E-01 .3711895E-01 -.3651500E-01 .3353199E-01 .2849854E-01 .2194075E-01 .1485414E-01 -.8441447E-02 .4088452E-02 .2573512E-02 .3018232E-02 .4343767E-02 -.5130981E-02 .4430971E-02 .1953150E-02 .2169221E-02 .7259721E-02 .1254239E-01 .1731151E-01 .2101175E-01 .2337672E-01 .2408515E-01 .2318889E-01 .2091309E-01 .1798833E-01 .1512700E-01 .1254632E-01 .1028396E-01 .8786912E-02 .8199611E-02 .8162864E-02 .8238027E-02 .8113801E-02 .781785E-02 .5599101E-02 .2630334E-02 .1180531E-02 -.5193853E-02 .8605611E-02 .1074382E-01 .1090159E-01 .8624530E-02 -.5048349E-02 .1544733E-02 .1564154E-02 .4149710E-02 .5825668E-02 .6809368E-02 .7362948E-02 .7545719E-02 .7202183E-02 .6376175E-02 .5509009E-02 .4656390E-02 .3727025E-02 .2821958E-02 .1502586E-02 -.6669071E-03 .3841629E-02 .7724360E-02 .1164601E-01 .1456786E-01 -.1570573E-01 .1478819E-01 .1223283E-01 .8367535E-02 .3606042E-02 .1347761E-02 .5711060E-02 .9663458E-02 .1305416E-01 .1516586E-01	.8526814E-02 .4636873E-01 .2439239E-01 .2045557E-01 .1174061E-01 -.1158999E-01 .8126689E-01 .6667212E-01 .2789299E-01 .1196284E-01 -.1675051E-01 .3814395E-01 .2346663E-01 .2358613E-01 .2279536E-01 .2760839E-01 .2007906E-01 .2270983E-01 .1163736E-01 .7224054E-02 -.3732142E-01 .6124948E-01 .5055979E-01 .6922311E-02 .4055962E-01 .3141268E-01 .3511185E-01 .2627182E-01 .1521222E-01 .2371242E-01 -.1277507E-01 .1539847E-01 .4164997E-01 .2734254E-01 .1715621E-01 .1464376E-01 .9880234E-02 .6116528E-01 .5633757E-01 .1781326E-03 .6419496E-01 .1007796E-01 .7063588E-01 .4978383E-01 .2097521E-01 .8435808E-01 .6002224E-01 .7937513E-02 .1148831E-01 .1726480E-01 -.1642531E-01 .1625736E-01 .6462529E-02 .2513505E-01 .3059427E-01 .4282703E-02 .9116567E-02 .3900878E-01 .6126064E-03 .5340668E-01 .7061285E-01 .1010951E-01 .6095846E-01 .1522451E-01 .3455323E-01 .5127202E-01 .1278236E-02 .4348042E-01 .2890754E-01 .1561617E-01 .2139704E-01 .4538726E-01 .6775159E-01 .5977420E-01 .1640026E-01 -.3187191E-01 .1011942E-01 .2348569E-02 .9211569E-02 .2709866E-01 .4553992E-01 .7897239E-02 .2578654E-01 .3839286E-01 .3578264E-01 -.4717434E-02 .6106469E-02 .4312431E-01 .3505096E-01 .1056666E-01 -.6852084E-02 .2405711E-01 .1708843E-01 .3195467E-01 .1107085E-01 .3538957E-01 .1208093E-01 .2307918E-02 .4335720E-01 .4845547E-01 -.4756534E-02 .2425945E-01 .1706756E-01 .2819323E-01 .1699670E-01 -.6657077E-01 .3603148E-01 .1751400E-01 .2943979E-02 .9095591E-02 -.2238128E-01 .3277542E-01 .2299671E-01 .2530983E-01 .2091850E-01 .2635352E-01 .5743748E-02 .6939079E-01 .7206349E-01 .7748706E-02 .2337797E-03 .1098232E-01 .7604971E-02 .8102413E-03 .4578796E-01 .5881239E-01 .1744656E-01 .3426799E-01 .2291130E-01 .1387471E-01 .7317761E-02 .8205648E-02 .1793881E-01 .1000294E-02 .1830016E-01 -.1294236E-01 .2250627E-01 .6188949E-02 .5939392E-01 .8670045E-01 .3508141E-01 .2571853E-01 .4122567E-01 .1633852E-01 .7544530E-02 -.4228194E-01 .2537692E-01 .4101631E-01 .5934445E-01 .4422836E-02 .4133255E-01 .9553500E-02 .4874093E-02 .1941227E-02 .5868308E-01 -.6814709E-01 .2931538E-01 .1284563E-01 .1206728E-01 .1516239E-01 -.1154765E-01 .3969730E-01 .2489185E-01 .4936257E-01 .7683827E-01 .4499426E-01 .2937598E-01 .1179044E-01 .2827755E-01 .1227652E-01 .3965096E-01 .1362194E-01 .1361604E-01 .1388364E-02 .1377707E-01 .4866447E-01 .2823223E-01 .4798098E-02 .1628104E-01 .3483088E-01 .2856987E-01 .5286204E-02 .1550431E-01 .2389600E-01 .1799730E-01 -.2107465E-01 .5590170E-01 .3937587E-01 .1431659E-01 .2750226E-01 .1595395E-01 .4967569E-01 .6211074E-01 .5320527E-01 .6508710E-01 -.5857846E-01 .4724647E-01 .4448505E-01 .7605040E-01 .5172641E-01 -.2136794E-03 .9386442E-02 .4840327E-02 .2537718E-01 .1301940E-01 -.5021705E-02 .1128869E-01 .6494265E-02 .1846480E-02 .3436992E-02 -.2029684E-01 .1832510E-01 .7461693E-02 .6718242E-02 .3066658E-01 .5317016E-01 .4823734E-01 .5042150E-02 .4240843E-01 .5665828E-01 -.5077621E-01 .2418051E-01 .4038142E-01 .7882125E-01 .4604644E-01 .2097188E-01 .5567687E-01 .3970771E-01 .5849985E-02 .2646152E-01 -.2316594E-01 .2087302E-01 .3912066E-01 .3878515E-01 .6759519E-01 .5614790E-01 .1815027E-01 .3448233E-01 .2823735E-01 .4586226E-01 .2113381E-01 .1003725E-01 .1357974E-01 .6236930E-01 .3744273E-01 .2273037E-01 .7978006E-02 .5117407E-01 .4264898E-01 .3495466E-01 .2267472E-01 .2239355E-01 .2276288E-01 .1479025E-01 .2717130E-02 -.1618160E-01 .3880700E-01 .1971704E-01 .1861827E-01 .3019903E-01 .1390011E-01 .1781640E-01 .1069248E-01 .2590228E-01 .6334368E-01 .4151352E-01 .7137437E-01 .9777783E-01 .3148390E-01 .1374946E-01 .2629209E-02 .6005804E-01 .5065469E-01 .2258359E-01 .4893523E-01 -.3759693E-01 .3206455E-01 .1898097E-01 .6592398E-01 .8100394E-01 .5841054E-01 .1314721E-01 .2109341E-01 .1043812E-01 .1736119E-01 -.1294642E-01 .3894668E-01 .3171899E-01 .7336407E-02 .1414052E-01 .4982901E-02 .9145109E-02 .4040915E-01 .2837569E-01 .2629437E-01 -.5382020E-01 .3838472E-01 .6276371E-02 .2504402E-02 .1355767E-01 -.6448434E-03 .2381181E-01 .2921245E-01 .1284711E-01 .1178421E-01 .3034780E-01 .3479945E-01 .3786312E-01 .6706497E-01 .8023492E-01 .7577585E-01 .8432457E-01 .8112141E-01 .7261312E-01 .7566528E-01 .6962194E-01 .6350127E-01 .7299923E-01 .5467446E-01 .4978925E-02

APPENDIX I: PEER Ground Motion Data Base RSN454_MORGAN_GIL337.AT2

.1573050E-01 .1470587E-01 .1252763E-01 .9885178E-02 .7338840E-02
.5327747E-02 .3791065E-02 .2084060E-02 .2107922E-03 .2802007E-02
-4.785425E-02 .6004741E-02 .6851918E-02 .7410012E-02 .7883480E-02
-9.006090E-02 .1093108E-01 .1285332E-01 .1449830E-01 .1573295E-01
-1.682876E-01 .1733373E-01 .1692526E-01 .1598550E-01 .1451060E-01
-1.241265E-01 .9872697E-02 .6983095E-02 .3459622E-02 .1499921E-03
.2986685E-02 .4661182E-02 .4962470E-02 .3719860E-02 .1238435E-02
-1.951253E-02 .5099003E-02 .7855573E-02 .1019236E-01 .1189340E-01
-1.276413E-01 .1302165E-01 .1322483E-01 .1409568E-01 .1544775E-01
-1.705668E-01 .1830661E-01 .1879793E-01 .1872543E-01 .1850792E-01
-1.832780E-01 .1788616E-01 .1688471E-01 .1509467E-01 .1244215E-01
-8.647282E-02 .3988181E-02 .4426235E-03 .4100962E-02 .6578127E-02
.7699997E-02 .7899483E-02 .7826456E-02 .7879114E-02 .7922579E-02
.7221009E-02 .5578443E-02 .2882565E-02 .7328048E-03 .4707624E-02
-8.266231E-02 .1095854E-01 .1223059E-01 .1141418E-01 .1428951E-02
-3.585121E-02 .1032430E-02 .5038954E-02 .7946065E-02 .9655644E-02
.1040637E-01 .1044516E-01 .9434318E-02 .7095280E-02 .3834662E-02
-1.495937E-03 .4351711E-02 .8028905E-02 .1078767E-01 .1219905E-01
-1.210353E-01 .1090904E-01 .8992402E-02 .6653884E-02 .4140218E-02
-1.581288E-02 .9862126E-03 .3593592E-02 .6177255E-02 .8398452E-02
.9997617E-02 .1063801E-01 .9824145E-02 .7692711E-02 .4936629E-02
.2184776E-02 .8283706E-04 .1000162E-02 .2969171E-03 .1469190E-02
.3009865E-02 .3585434E-02 .3211305E-02 .1992721E-02 .9652861E-03
-8.029980E-03 .1770306E-02 .2111045E-02 .2141493E-02 .2426012E-02
-2.940353E-02 .3460818E-02 .4072519E-02 .4600996E-02 .4754068E-02
-4.236370E-02 .3103548E-02 .1676271E-02 .5232050E-03 .5787828E-03
-1.662282E-02 .3291782E-02 .4586490E-02 .5018349E-02 .4731585E-02
-3.915090E-02 .2853273E-02 .2099520E-02 .1904236E-02 .2038191E-02
-2.183435E-02 .2101146E-02 .1496776E-02 .4388526E-03 .4561424E-03
.8131327E-03 .5821798E-03 .4179425E-03 .1853811E-02 .3921957E-02
-6.471810E-02 .8778221E-02 .1029015E-01 .1051611E-01 .9496367E-02
-7.623499E-02 .5647113E-02 .4080663E-02 .3071832E-02 .2904688E-02
-3.163692E-02 .3045404E-02 .2355406E-02 .6474860E-03 .2603796E-02
.7082267E-02 .1251828E-01 .1850006E-01 .2391941E-01 .2757824E-01
.2846124E-01 .2693132E-01 .2352126E-01 .1888392E-01 .1386024E-01
.9323316E-02 .5784742E-02 .3042301E-02 .6371374E-03 .1487377E-02
-3.286813E-02 .4637284E-02 .5106102E-02 .4796239E-02 .3936024E-02
-2.755723E-02 .1465299E-02 .9483000E-04 .1673331E-02 .3899938E-02
.5979489E-02 .7297594E-02 .7848472E-02 .7300509E-02 .5795834E-02
.3588305E-02 .7174126E-03 .2385858E-02 .5108435E-02 .6711094E-02
-6.952046E-02 .5764323E-02 .3323834E-02 .3756541E-03 .5097627E-02
.1035000E-01 .1550841E-01 .1966274E-01 .2230970E-01 .2372515E-01
.2426091E-01 .2400998E-01 .2304490E-01 .2133437E-01 .1901106E-01
.1629688E-01 .1301564E-01 .9021140E-02 .4941658E-02 .1169863E-02
-1.916847E-02 .3801149E-02 .4650250E-02 .4479930E-02 .3138732E-02
-8.811468E-03 .1773143E-02 .4246398E-02 .5978158E-02 .7263750E-02
.8172911E-02 .8737527E-02 .9330036E-02 .9823400E-02 .9579267E-02
.8307052E-02 .6690876E-02 .4922996E-02 .2998757E-02 .1355405E-02
-1.827105E-03 .1743275E-02 .3178729E-02 .4299919E-02 .5451092E-02
-6.456209E-02 .6996495E-02 .6922554E-02 .5825852E-02 .3696360E-02
-9.877612E-03 .1696638E-02 .3799587E-02 .5178632E-02 .5662264E-02
.4727166E-02 .2150507E-02 .1580909E-02 .5691320E-02 .9411092E-02
-1.187852E-01 .1338196E-01 .1424783E-01 .1447602E-01 .1364222E-01
-1.168671E-01 .9082639E-02 .6148814E-02 .2745702E-02 .4689197E-03
.3059584E-02 .4973222E-02 .6414429E-02 .7532992E-02 .8058521E-02
.8262597E-02 .8048201E-02 .7251111E-02 .6012735E-02 .4533676E-02
.2880035E-02 .1604989E-02 .9603218E-03 .5355739E-02 .2503551E-03
.3856108E-03 .7570974E-03 .1032257E-02 .1118968E-02 .1086174E-02
.1034586E-02 .8402485E-03 .3272023E-04 .1087709E-02 .2243000E-02
-3.319058E-02 .4020933E-02 .4254833E-02 .4309969E-02 .4758148E-02
-5.729508E-02 .6868078E-02 .8172849E-02 .9415659E-02 .1071890E-01
-1.178047E-01 .1247257E-01 .1336985E-01 .1433626E-01 .1494646E-01
-1.503264E-01 .1446973E-01 .1326538E-01 .1127739E-01 .8334834E-02
-4.729605E-02 .1215744E-02 .1754675E-02 .4211893E-02 .6087036E-02

APPENDIX II: PEER Ground Motion Data Base RSN1633_MANJIL_ABBAR--L.AT2

-1.165973E-01 .3512507E-01 .5315885E-01 .6424719E-01 .6235324E-01
.3999089E-01 .2028477E-01 .1331182E-01 .4208875E-01 .7031680E-01
-4.614632E-01 .4469707E-01 .7137175E-01 .3460430E-02 .4210115E-01
.1113423E-02 .6490055E-02 .1423914E-01 .7866856E-02 .6486088E-01
-9.559631E-01 .7440250E-01 .6099845E-01 .6977577E-01 .1150147E+00
-1.401281E+00 .1213274E+00 .1000772E+00 .1120965E+00 .1255078E+00
-1.217934E+00 .8503521E-01 .3064579E-01 .1413055E+00 .1423148E+00
.6361420E-01 .1299742E-01 .4099214E-02 .3147716E-02 .4630582E-01
.3561450E-01 .2251036E-01 .7137182E-01 .7925611E-01 .1146492E+00
.1255240E+00 .8896165E-01 .5785352E-01 .1078947E-01 .2750384E-01
-3.895962E-01 .4839349E-01 .2703818E-01 .3605187E-02 .2096984E-01
.7161005E-01 .3849518E-01 .5759177E-01 .6026430E-01 .2196136E-01
.1959340E-01 .9342942E-01 .1268993E+00 .7406288E-01 .1646034E-01
-2.928677E-01 .2993740E-01 .4735740E-01 .7552791E-02 .1068519E-02
.3684911E-01 .3440731E-01 .2079861E-01 .1542915E-02 .5782834E-01
-1.181923E+00 .1193932E+00 .7290801E-01 .1179151E-01 .1233334E+00
.7356047E-01 .1645009E-01 .1602529E-01 .6446528E-02 .5790870E-01
-4.826904E-01 .4106590E-01 .7035843E-01 .5843128E-01 .2515847E-01
.2066786E-01 .1980332E-01 .1495246E-01 .3138181E-01 .5486829E-02
.3296978E-01 .3241194E-01 .2262830E-01 .8197872E-01 .3409243E-02
.7982256E-01 .1216805E+00 .1203725E+00 .2660184E-01 .6393565E-01
-7.677779E-01 .8237881E-01 .7814989E-01 .4256944E-01 .7588559E-03
.1016397E+00 .1021160E+00 .1269163E-01 .5620901E-01 .1119458E-02
.1888776E-01 .2399608E-01 .5225339E-01 .6705192E-01 .4372464E-01
-3.593087E-01 .3301018E-01 .2187887E-02 .2543460E-02 .2507448E-01
-2.832889E-01 .2082291E-01 .2059604E-01 .1662242E-02 .2098442E-01
.2663524E-01 .3208536E-01 .6251498E-01 .9919176E-01 .1425550E+00
.1201211E+00 .3269967E-01 .2516805E-01 .1716039E-01 .5439084E-01
-1.064720E+00 .1474605E+00 .1961142E+00 .2369260E+00 .2311001E+00
-1.577254E+00 .6675083E-01 .1713310E-01 .1347379E+00 .1518056E+00
.8348106E-01 .1084668E+00 .1179037E+00 .1491698E+00 .1006062E+00
-4.929566E-01 .1217748E+00 .1406743E+00 .1315290E+00 .1191924E+00
-7.292621E-01 .5578874E-02 .3460241E-01 .3384232E-01 .3166407E-01
.9224299E-01 .1026481E+00 .1726060E-01 .2158525E-01 .2855048E-01
-6.247412E-01 .4302995E-01 .5011557E-01 .1016641E+00 .1034128E+00
-7.727495E-01 .7887855E-01 .6593779E-01 .1238143E-01 .2085495E-01
-1.724562E-03 .3523242E-01 .4179182E-01 .1216927E-01 .1743575E-02
.2068359E-02 .1715789E-01 .1608394E-01 .1374439E-01 .1965480E-01
-2.755086E-02 .1687724E-01 .1036408E-01 .8070989E-02 .1450376E-01
-5.872725E-01 .9777356E-01 .8609869E-01 .1948305E-01 .6117363E-02
.3100952E-01 .8971365E-01 .6420663E-01 .1224901E-01 .1614198E-01
.2838174E-01 .3843253E-02 .1164288E-02 .2694838E-01 .2465726E-01
-1.829463E-01 .5042002E-01 .2918736E-01 .5435583E-02 .5648930E-02
-1.254622E-01 .4535318E-01 .3940493E-01 .2851602E-01 .3577247E-01
-2.656281E-01 .3264535E-01 .2026597E-01 .2831505E-01 .2722742E-01
-6.336862E-02 .2578326E-01 .5142076E-01 .3243615E-01 .2734688E-02
-5.206903E-03 .2685677E-01 .2155223E-01 .2925698E-01 .8126712E-02
.4158561E-01 .1988371E-01 .1625626E-01 .1248318E-01 .3388937E-01
.6595856E-01 .5877278E-01 .2971969E-01 .7095153E-02 .8040477E-02
-1.330225E-01 .1825993E-01 .5034198E-01 .2851235E-01 .1599768E-01
-4.987471E-01 .6705473E-01 .1781311E-01 .1593338E-01 .4024962E-01
.3002265E-02 .1388668E-01 .1315435E-01 .1918162E-01 .1499048E-01
.4462836E-02 .1286104E-01 .6466147E-01 .7152567E-01 .3081400E-01
.2942376E-02 .2408103E-01 .4227822E-02 .2286936E-01 .1302720E-02
-2.965318E-01 .1472520E-01 .1757011E-01 .3403444E-02 .1739239E-01
.1096346E-02 .2428705E-01 .2130523E-01 .2136756E-02 .2957146E-01
.3843083E-01 .2769640E-02 .1138721E-01 .1656540E-01 .3179954E-01
.4013056E-01 .6041454E-01 .7500385E-01 .4258450E-01 .1698433E-01
-4.215313E-01 .1459397E-01 .4174526E-02 .2231959E-01 .1725699E-01
.6450313E-01 .6361882E-01 .4387615E-01 .1919402E-01 .1381511E-02
-3.178949E-01 .4650994E-01 .2363180E-01 .2065832E-01 .3584485E-01
-3.690842E-01 .1656059E-01 .1002480E-01 .2511999E-01 .1909707E-01
.1237390E-01 .1731568E-01 .1198197E-01 .6444121E-01 .7685871E-01
.4531028E-01 .4115722E-01 .5745348E-01 .4705911E-01 .1795692E-01

APPENDIX I: PEER Ground Motion Data Base RSN454_MORGAN_GIL337.AT2	APPENDIX II: PEER Ground Motion Data Base RSN1633_MANJIL_ABBAR--L.AT2
.7061901E-02 .6900393E-02 .5779595E-02 .3895364E-02 .1599964E-02 -.4753267E-03 -.1955475E-02 -.2678821E-02 -.2684626E-02 -.2014129E-02 -.8806814E-03 .5579815E-03 .2323277E-02 .4096469E-02 .5486350E-02 .6187286E-02 .6314795E-02 .6220364E-02 .6035915E-02 .5499591E-02 .4633659E-02 .3927245E-02 .3644855E-02 .3681962E-02 .3801335E-02 .3668974E-02 .2628916E-02 .4992832E-03 -.2272959E-02 -.5138668E-02 -.7727952E-02 -.9731103E-02 -.1092480E-01 -.1082592E-01 -.9833335E-02 -.8228250E-02 -.6280143E-02 -.4830898E-02 -.4305894E-02 -.4407614E-02 -.4444195E-02 -.3648687E-02 -.2443676E-02 -.1607920E-02 -.1290852E-02 -.9342760E-03 -.3770765E-03 -.4328059E-03 -.1047366E-02 -.1614482E-02 -.1826404E-02 -.1932210E-02 -.2617122E-02 -.3791556E-02 -.5314919E-02 -.6760843E-02 -.8236234E-02 -.9585332E-02 -.1021165E-01 -.1033832E-01 -.9898865E-02 -.8616894E-02 -.6686070E-02 -.4089728E-02 -.1114215E-02 .1978679E-02 .5417963E-02 .8735754E-02 .1168803E-01 .1366375E-01 .1441729E-01 .1422631E-01 .1313857E-01 .1158332E-01 .1006662E-01 .8775383E-02 .776078E-02 .7004012E-02 .6729749E-02 .6713802E-02 .6744308E-02 .6535415E-02 .5443037E-02 .3291777E-02 .5239244E-03 -.2329213E-02 -.4911390E-02 -.6726703E-02 -.7152854E-02 -.6108777E-02 -.3716171E-02 -.7042224E-03 .2197147E-02 .4684098E-02 .6591363E-02 .8045382E-02 .9341119E-02 .1060847E-01 .1146100E-01 .1137286E-01 .1036534E-01 .8881346E-02 .7303900E-02 .5481810E-02 .3439306E-02 .1716733E-02 .5755850E-04 -.1862238E-02 -.3624844E-02 -.5215355E-02 -.6541123E-02 -.7868102E-02 -.8973004E-02 -.9564188E-02 -.9730391E-02 -.9548632E-02 -.9161038E-02 -.9076748E-02 -.9367905E-02 -.9802859E-02 -.1049567E-01 -.1145928E-01 -.1234107E-01 -.1307677E-01 -.1347408E-01 -.1326433E-01 -.1278708E-01 -.1238597E-01 -.1192847E-01 -.1141798E-01 -.1088477E-01 -.1066995E-01 -.1059980E-01 -.1068479E-01 -.1058022E-01 -.1025490E-01 -.9960240E-02 -.9746441E-02 -.9266729E-02 -.8391253E-02 -.7406553E-02 -.6198830E-02 -.5050159E-02 -.4125507E-02 -.3469927E-02 -.2849627E-02 -.2310582E-02 -.1905813E-02 -.1283110E-02 -.1320205E-03 .1644454E-02 .3699391E-02 .5799634E-02 .7581989E-02 .8593003E-02 .8846263E-02 .8740292E-02 .8559681E-02 .8125323E-02 .7355411E-02 .6737177E-02 .6892636E-02 .7804412E-02 .8856615E-02 .9819093E-02 .1080668E-01 .1194826E-01 .1316912E-01 .1406059E-01 .1441971E-01 .1431600E-01 .1369214E-01 .1270944E-01 .1172728E-01 .1038616E-01 .8362246E-02 .5788063E-02 .2996067E-02 .4228722E-03 -.1389314E-02 -.2737793E-02 -.3586634E-02 -.1369203E-02 -.2719195E-02 -.1701462E-02 -.5120644E-03 .4695826E-03 .9097797E-03 .8247944E-03 .1819982E-03 -.8154474E-03 -.1599261E-02 -.1877856E-02 -.1718540E-02 -.9726360E-03 .5376162E-03 .2467305E-02 .4650293E-02 .7045703E-02 .9497404E-02 .1160353E-01 .1310830E-01 .1388190E-01 .1408170E-01 .1386793E-01 .1302447E-01 .1185800E-01 .1020371E-01 .7661186E-02 .4666536E-02 .2039676E-02 -.1005420E-03 -.1840571E-02 -.3127898E-02 -.3563291E-02 -.3272352E-02 -.2483763E-02 -.1369203E-02 -.2105545E-03 .7352335E-03 .1803474E-02 .2702555E-02 .2755928E-02 .1936062E-02 .9999076E-03 .5437535E-03 .6797275E-03 .1437309E-02 .2723394E-02 .4370675E-02 .5673943E-02 .6302242E-02 .6673571E-02 .7093188E-02 .7777904E-02 .8482978E-02 .8796935E-02 .8355476E-02 .7314268E-02 .6376527E-02 .5802511E-02 .4850324E-02 .2971699E-02 .5178642E-03 -.1555130E-02 -.3302073E-02 -.5257517E-02 -.7491808E-02 -.9231718E-02 -.9993103E-02 -.9927559E-02 -.9224339E-02 -.8103222E-02 -.6829799E-02 -.5554403E-02 -.4400864E-02 -.3567564E-02 -.2894487E-02 -.2372348E-02 -.2190589E-02 -.2530990E-02 -.3158044E-02 -.3918551E-02 -.4507997E-02 -.4729747E-02 -.4672193E-02 -.4569993E-02 -.4647046E-02 -.5215652E-02 -.6245118E-02 -.7155866E-02 -.7639008E-02 -.800939E-02 -.8332808E-02 -.8552751E-02 -.8554396E-02 -.8150506E-02 -.7721316E-02 -.7914712E-02 -.8782541E-02 -.9695779E-02 -.1013194E-01 -.1009325E-01 -.9875533E-02 -.9646986E-02 -.9161217E-02 -.8313369E-02 -.7526412E-02 -.7034270E-02 -.6491831E-02 -.5663006E-02 -.4775330E-02 -.3841639E-02 -.2749362E-02 -.1483079E-02 -.3220344E-04 .1771281E-02 .3666249E-02 .5422357E-02 .7216148E-02 .8988921E-02 .1048798E-01 .1121918E-01 .1068859E-01 .9596303E-02 .8688468E-02 .7761792E-02 .6975306E-02 .6287571E-02 .5130409E-02 .3534772E-02 .1721265E-02 .9646820E-04 -.1568540E-02 -.2888061E-02 -.3799226E-02 -.4171556E-02 -.3917992E-02 -.2639286E-02 -.3759686E-03	.1844381E-01 .3140963E-01 .3904863E-01 .5236220E-02 -.4151915E-01 -.5668366E-01 -.1297787E-01 .1355039E-01 .3001827E-01 .8585900E-01 .9303421E-01 .4832726E-01 .1998629E-01 .1306860E-01 -.2913263E-01 -.2432124E-01 .1031204E-01 .3295678E-02 .4438313E-01 .5174532E-01 -.5401351E-02 -.3296472E-01 -.4230639E-04 .2957737E-01 -.1310759E-03 -.4076624E-01 -.2472558E-01 .4811445E-01 .4894339E-01 .1250217E-01 -.2241104E-02 -.6015639E-03 .2848796E-01 .1809032E-03 -.4226917E-01 -.4429915E-01 -.2043702E-01 -.1831843E-01 -.3559601E-01 -.5764619E-02 .3785670E-02 -.2082628E-01 -.4471709E-01 -.5142865E-01 -.4375760E-01 -.5194080E-01 -.6713781E-01 -.5645568E-01 -.1926854E-01 .5639849E-02 .3976594E-01 .4426756E-01 -.5107599E-02 -.5751296E-01 -.5753774E-01 -.1181931E-01 .1862644E-01 .6065643E-01 .9811546E-01 .7439518E-01 .1616087E-01 -.2250111E-01 -.1058672E-01 .4542708E-02 -.2773343E-01 -.3227859E-01 -.1730653E-02 -.3034404E-01 -.5311058E-01 -.2253217E-01 .2282248E-01 .4363816E-01 .2005223E-01 .1170639E-01 .3794442E-01 .2688471E-01 -.4231729E-01 -.4658465E-01 .4387369E-02 .4669420E-02 .3039231E-01 .3407520E-01 -.1235029E-01 -.3768895E-01 -.1044574E-01 .2384288E-01 .4318343E-01 .5185675E-01 .4435717E-01 .1433970E-01 -.3069172E-02 .1648923E-01 .2661044E-01 .5400191E-02 -.4011501E-02 -.5975921E-02 -.4754473E-01 -.8492578E-01 .8105372E-01 -.6511633E-01 -.6679525E-01 -.6670208E-01 -.7405254E-01 -.6597700E-01 -.3632950E-01 -.2917287E-01 .3903645E-01 .3172533E-01 .2008064E-01 .3986536E-01 .2755566E-01 .2745167E-01 .3809507E-01 .6176872E-01 .4179932E-01 .8822474E-02 .2180004E-03 -.1628271E-01 .9830235E-02 -.5542582E-02 -.3493922E-01 -.4775569E-01 .3892470E-01 .1028423E-01 .2013913E-01 .2463758E-01 .7363388E-02 .1974464E-01 .3326464E-01 .1612141E-01 .1027362E-01 .2676282E-01 .3266301E-01 .4591129E-01 .3927517E-01 -.1272500E-01 -.1754874E-01 .2134625E-01 .3220204E-01 .9393773E-02 -.2075918E-02 .1165105E-01 .2594084E-01 .2181941E-01 .1482642E-01 -.3511722E-01 -.2392540E-03 .4677737E-01 .2473456E-01 .7173662E-02 -.2531912E-01 -.2101396E-01 .8807969E-02 .4297552E-01 .2822853E-01 .1659180E-01 .9997303E-02 .3460853E-01 .5607610E-01 .4720840E-01 -.1656172E-01 .1481581E-01 .3163655E-02 .3957820E-01 .8530679E-02 -.2284144E-01 .6008430E-02 .1629777E-01 .2968941E-01 .8444411E-02 .1119624E-02 .6614652E-02 .1505928E-01 .2223584E-01 .6197754E-01 -.5504533E-01 .3197328E-02 .8993942E-03 .2107834E-01 .4152633E-01 -.5840784E-01 .4044076E-01 .1530525E-01 .1947225E-01 .2803206E-01 .1131334E-01 .2309604E-01 .1130484E-01 .6932928E-01 .5345939E-01 .2325025E-01 .6545061E-01 .5245369E-01 .3849436E-01 .3426292E-01 .1508255E-01 .1718345E-01 .3659822E-01 .1687019E-01 .3772389E-01 .1321519E-01 .2157706E-01 .2650351E-01 .2457197E-01 .3938537E-02 .1258341E-01 .1655496E-01 .3031240E-01 .4252071E-01 .1086905E-01 .3882370E-01 .1009097E-01 .2332658E-01 .1732190E-01 .2634903E-02 -.1783261E-01 .1844337E-01 .1714320E-01 .2334851E-01 .1993009E-02 .6481700E-02 .2679797E-01 .1977897E-01 .2454696E-02 .1499984E-01 .1914761E-02 .4520322E-01 .3094070E-01 .9145708E-02 .1699759E-01 -.1966543E-01 .2009073E-01 .2308871E-01 .1468718E-01 .1065973E-01 -.2346156E-01 .3270405E-01 .2976925E-01 .3258986E-01 .1127118E-01 .5752978E-01 .8146575E-01 .8750954E-01 .6674591E-01 .3211966E-01 .2396573E-02 .1626229E-02 .3304286E-01 .4934665E-01 .1465531E-01 -.1350081E-01 .7104982E-02 .3093154E-02 .1866054E-02 .4460301E-02 -.1133396E-01 .2922793E-01 .7016276E-02 .1150550E-01 .1163843E-01 -.1751440E-01 .4944138E-01 .2443537E-01 .1531473E-01 .2512657E-01 .2330559E-02 .6281781E-02 .4635362E-02 .2277511E-01 .3932665E-01 .4961526E-01 .3207878E-01 .3656235E-01 .5300170E-01 .4592690E-02 .3923570E-01 .5890614E-01 .9503635E-01 .7676966E-01 .3107518E-01 -.7889271E-01 .4219055E-01 .9000613E-02 .1490807E-01 .5210071E-01 -.4169713E-01 .1166992E-01 .7704579E-02 .4334980E-01 .4932011E-01 .3106194E-02 .2519064E-02 .3014645E-01 .2031742E-01 .2303552E-02 -.1626573E-01 .1045664E-01 .2988726E-01 .2083664E-01 .9051541E-02 .1767404E-02 .1626651E-01 .8246166E-02 .6255538E-02 .3064338E-01 -.1248541E-01 .6180075E-01 .2614440E-01 .2028781E-01 .1140808E-02 -.3689144E-01 .1361282E-01 .1595808E-01 .3067466E-01 .4312478E-01 .5733038E-01 .5358553E-01 .5589944E-02 .2133708E-01 .8305708E-02

APPENDIX I: PEER Ground Motion Data Base RSN454_MORGAN_GIL337.AT2

.2381161E-02 .5165992E-02 .7500574E-02 .8918006E-02 .9556248E-02
.9456022E-02 .8822373E-02 .7701655E-02 .6471205E-02 .5148703E-02
.3905515E-02 .2297137E-02 .3399979E-03 .1409670E-02 .2930243E-02
.3894122E-02 .4211486E-02 .4152731E-02 .4068682E-02 .4105570E-02
.4074634E-02 .3536081E-02 .2557382E-02 .1618244E-02 .7053324E-03
.2116178E-03 .5500642E-03 .1706729E-03 .1483938E-02 .3045388E-02
.4266206E-02 .5183942E-02 .5994175E-02 .6666365E-02 .6673918E-02
.6246954E-02 .5730416E-02 .5330166E-02 .5012278E-02 .5247629E-02
.6041819E-02 .6784708E-02 .6943545E-02 .6381979E-02 .5532278E-02
.4698422E-02 .4206319E-02 .3807090E-02 .2934368E-02 .1837121E-02
.7894219E-03 .2962884E-03 .6094513E-03 .1189809E-02 .1529399E-02
.1527449E-02 .1354715E-02 .1073153E-02 .4389622E-03 .4874103E-03
.1204686E-02 .1466537E-02 .1405249E-02 .1275414E-02 .1373313E-02
.2029381E-02 .3070356E-02 .4027556E-02 .4940841E-02 .5834348E-02
.6579493E-02 .7283396E-02 .7644119E-02 .7370860E-02 .6574007E-02
.5300913E-02 .3969551E-02 .2482957E-02 .9343908E-03 .1371052E-03
.1104796E-02 .1140894E-02 .7428046E-03 .6461966E-03 .7834742E-03
.5309664E-03 .2467347E-03 .1178317E-02 .2186659E-02 .3167374E-02
.3786920E-02 .3967961E-02 .3905623E-02 .3825870E-02 .3831332E-02
.3763539E-02 .3236908E-02 .2311903E-02 .1393573E-02 .4514216E-03
.4930006E-03 .1409377E-02 .2577404E-02 .3569237E-02 .4190998E-02
.4725831E-02 .5287796E-02 .5708880E-02 .5192652E-02 .3807238E-02
.2287494E-02 .1050324E-02 .3136705E-03 .1800579E-02 .3080963E-02
.3774465E-02 .3909479E-02 .3795901E-02 .3728809E-02 .3772128E-02
.3849813E-02 .3898729E-02 .3899746E-02 .3860416E-02 .3603433E-02
.2865801E-02 .1896327E-02 .1219839E-02 .1028139E-02 .1337296E-02
.2182505E-02 .3398783E-02 .4745151E-02 .6055907E-02 .7126974E-02
.7871180E-02 .8090675E-02 .7463683E-02 .6257770E-02 .4778460E-02
.3392307E-02 .1715718E-02 .5081600E-03 .3141154E-02 .5528131E-02
.7256195E-02 .8946716E-02 .1070581E-01 .1191587E-01 .1237542E-01
.1216203E-01 .1130977E-01 .1006279E-01 .8702697E-02 .7170923E-02
.5179030E-02 .2919751E-02 .9269491E-03 .8051190E-03 .2572496E-02
.4286281E-02 .5640929E-02 .6311540E-02 .6024462E-02 .4899382E-02
.3836145E-02 .2892039E-02 .1962181E-02 .1387347E-02 .1489867E-02
.2323716E-02 .3338159E-02 .3939588E-02 .4290818E-02 .4903635E-02
.5695522E-02 .5996685E-02 .5362262E-02 .3741821E-02 .1379765E-02
.1290685E-02 .3958095E-02 .6469076E-02 .8364504E-02 .9254570E-02
.9350598E-02 .8642482E-02 .7002465E-02 .5154306E-02 .3544512E-02
.2040782E-02 .6388433E-03 .4624113E-03 .1282494E-02 .2167950E-02
.3058109E-02 .3600922E-02 .3483642E-02 .2858153E-02 .2057650E-02
.1365764E-02 .1001354E-02 .6823997E-03 .1203475E-04 .9101551E-03
.1536409E-02 .1761495E-02 .1985817E-02 .2399297E-02 .2545558E-02
.2477547E-02 .2315644E-02 .1943348E-02 .1089575E-02 .3702214E-03
.2061353E-02 .3863169E-02 .5424672E-02 .6250258E-02 .6409614E-02
.6236778E-02 .5584950E-02 .4526757E-02 .3178016E-02 .1344836E-02
.4371427E-03 .1487898E-02 .2247760E-02 .3104180E-02 .3492240E-02
.2963795E-02 .1938374E-02 .5728725E-03 .7016510E-03 .1686404E-02
.2595827E-02 .3344669E-02 .3677376E-02 .3071906E-02 .1948992E-02
.1161605E-02 .9816951E-03 .1172161E-02 .1346521E-02 .1012108E-02
.4088039E-04 .1027439E-02 .1978796E-02 .2926848E-02 .3705592E-02
.4454170E-02 .5404199E-02 .6251376E-02 .6698277E-02 .6766025E-02
.6722779E-02 .6990190E-02 .7254904E-02 .7518475E-02 .7796908E-02
.7664851E-02 .7115356E-02 .5993905E-02 .4669395E-02 .3238745E-02
.1995469E-02 .1026376E-02 .2856357E-04 .1170712E-02 .2512894E-02
.3561085E-02 .4019199E-02 .4025776E-02 .3891774E-02 .3793776E-02
.3458909E-02 .2650633E-02 .1691333E-02 .4764437E-03 .7810146E-03
.2230105E-02 .4205603E-02 .6336858E-02 .8096026E-02 .9093861E-02
.9310437E-02 .8818018E-02 .7737726E-02 .6066523E-02 .3879792E-02
.1720258E-02 .4181777E-04 .1695734E-02 .3128451E-02 .4531361E-02
.5754071E-02 .6428719E-02 .6587569E-02 .6508346E-02 .6437458E-02
.6424439E-02 .6122171E-02 .5313072E-02 .4429965E-02 .3938048E-02
.3793474E-02 .3286517E-02 .3010344E-02 .3340987E-02 .3823430E-02
.4072580E-02 .4063418E-02 .3935602E-02 .3769229E-02 .3062777E-02
.2026967E-02 .1235126E-02 .3236817E-03 .6565745E-03 .1273725E-02

APPENDIX II: PEER Ground Motion Data Base RSN1633_MANJIL_ABBAR--L.AT2

.1908958E-01 .4563893E-02 .6907749E-02 .5926332E-02 .2077523E-01
.4938144E-02 .5536568E-01 .4954917E-01 .1327515E-01 .4240008E-01
.1010726E-01 .4599403E-02 .2523390E-01 .3492730E-02 .4840349E-01
.5618551E-01 .3726489E-01 .4085902E-01 .3709254E-01 .1585149E-01
.4933311E-01 .4594673E-01 .2677700E-01 .1122470E-01 .1535678E-01
.1059616E-01 .9021966E-02 .2136636E-01 .9513037E-02 .1039392E-01
.1443753E-01 .3338000E-01 .1666807E-01 .1013221E-01 .1953512E-01
.1687597E-01 .2304526E-01 .1633899E-01 .1211728E-01 .6402822E-02
.4131443E-01 .2529266E-01 .1701599E-01 .1672225E-01 .1149779E-01
.7514119E-02 .2203933E-01 .3737463E-01 .4102564E-01 .3883717E-01
.5606272E-01 .5054742E-01 .1137257E-01 .3689351E-01 .1591207E-01
.4452537E-01 .2646912E-01 .3291628E-01 .1587231E-01 .4039493E-01
.4111874E-01 .5322813E-02 .1686485E-01 .2215238E-01 .1241331E-01
.1810146E-01 .6802925E-03 .8014374E-02 .3160895E-01 .4170479E-01
.3621991E-01 .2373040E-02 .3477936E-01 .4955214E-02 .4510370E-01
.4626296E-01 .1289102E-01 .4151136E-01 .2932080E-01 .1749333E-01
.8294685E-02 .1791833E-01 .3639635E-01 .9333029E-02 .2952497E-01
.1464667E-01 .1263794E-01 .1277070E-01 .2015729E-01 .1882938E-01
.2678388E-01 .2245574E-01 .1144697E-01 .7471519E-02 .2786365E-01
.1969915E-02 .8047838E-02 .2168274E-01 .2858073E-01 .8030814E-02
.1838949E-01 .3259773E-02 .2335729E-01 .3294460E-01 .3284002E-01
.1809203E-01 .1197662E-01 .1302240E-01 .2891458E-01 .2301277E-01
.2987163E-01 .2840509E-01 .1291093E-01 .2449527E-01 .28464598E-03
.2571379E-02 .1393018E-01 .2470063E-01 .3954242E-01 .4755669E-01
.3145249E-01 .3287368E-02 .8480754E-02 .5729306E-02 .1605646E-01
.2582670E-02 .5357074E-02 .3690234E-01 .3626558E-01 .2637769E-01
.3147984E-01 .2310231E-01 .6582335E-02 .4508679E-02 .1076770E-01
.4719601E-02 .1247628E-01 .1112122E-01 .1329160E-01 .3607371E-01
.1990736E-01 .5965671E-02 .9345655E-02 .1590736E-01 .3008955E-01
.2117358E-01 .1674005E-01 .3873957E-01 .2677717E-01 .3744015E-04
.2846078E-01 .4560680E-01 .6105385E-01 .3408721E-01 .5178478E-02
.1957587E-01 .3956792E-01 .1916329E-01 .7429783E-02 .6149306E-02
.3574965E-01 .3131057E-01 .1036003E-01 .1773243E-01 .4040193E-01
.2686833E-01 .2064039E-01 .1250524E-01 .7637081E-02 .2178127E-02
.7791934E-02 .3657433E-01 .5770686E-01 .1808459E-01 .7974148E-02
.1004067E-02 .1608732E-01 .3858494E-01 .3009142E-01 .8370236E-02
.2586458E-01 .2453982E-02 .3357686E-01 .2671935E-01 .1809219E-01
.3444720E-01 .3966292E-01 .6957787E-02 .1869441E-01 .1050467E-02
.2128385E-01 .3274137E-01 .1837554E-01 .3131718E-02 .3546274E-02
.2858676E-02 .2327272E-01 .2485115E-01 .5227839E-03 .1373795E-02
.1833984E-01 .2698985E-01 .7983170E-02 .9390072E-02 .3131660E-02
.1878008E-01 .1934038E-01 .5948269E-02 .2923462E-01 .2666235E-01
.1547163E-01 .1614312E-01 .4765267E-03 .2621171E-01 .2766044E-01
.6620420E-02 .1477213E-01 .6497443E-03 .367136E-01 .1679714E-01
.2438663E-01 .4573459E-01 .3296540E-01 .5928304E-02 .1304033E-02
.2569433E-01 .3546839E-01 .3996366E-01 .2652154E-01 .1069287E-01
.2387083E-01 .4237039E-01 .3626549E-01 .3581740E-01 .4947697E-01
.3394812E-01 .6392248E-03 .2085674E-01 .1731713E-01 .2850396E-02
.9707981E-02 .1891212E-01 .1262682E-01 .1213925E-01 .2660475E-01
.1556558E-01 .6882863E-02 .6197812E-02 .1550009E-01 .1264544E-01
.9965348E-02 .1849832E-01 .3969542E-01 .1023952E-01 .2062631E-01
.4455336E-01 .2744412E-01 .3390843E-02 .7177342E-03 .9594855E-02
.2945895E-02 .2464942E-02 .9404908E-03 .9118195E-02 .2039108E-01
.2772608E-01 .2712675E-01 .5889023E-02 .2436158E-01 .1505394E-01
.1096151E-01 .1712009E-01 .2204161E-01 .1825711E-01 .1055132E-01
.4491987E-02 .1164369E-01 .2181710E-01 .2880479E-01 .1240934E-01
.6306911E-02 .1082120E-01 .1643394E-01 .1513069E-02 .1015705E-01
.3182929E-02 .5771667E-02 .5837736E-02 .3510571E-02 .133596E-01
.7216191E-03 .9483021E-02 .1536054E-01 .2745332E-01 .1428489E-01
.1274987E-01 .1999613E-01 .4828395E-02 .1809267E-01 .3272247E-01
.5882796E-02 .1445420E-01 .1673657E-01 .4863570E-02 .4289621E-02
.1594257E-01 .1011215E-01 .2472751E-01 .4753427E-02 .1203373E-01
.4990549E-02 .1715692E-01 .1873904E-01 .1118503E-01 .2964247E-02
.9017922E-03 .8440039E-02 .1977311E-01 .1650772E-01 .9479163E-02

APPENDIX I: PEER Ground Motion Data Base RSN454_MORGAN_GIL337.AT2	APPENDIX II: PEER Ground Motion Data Base RSN1633_MANJIL_ABBAR--L.AT2
-.1448852E-02 -.1383568E-02 -.1302815E-02 -.1268746E-02 -.9491257E-03 -.1125624E-03 .7509382E-03 .7734158E-03 -.1365454E-03 -.1171412E-02 -.1691162E-02 -.1762115E-02 -.1853508E-02 -.1799913E-02 -.1592309E-02 -.1427196E-02 -.1297471E-02 -.8392246E-03 .6152595E-04 .9193664E-03 .1428697E-02 .1846979E-02 .2312974E-02 .2972971E-02 .3605318E-02 .3853553E-02 .3386398E-02 .2356006E-02 .1443049E-02 .1072411E-02 .1332504E-02 .2375123E-02 .3539897E-02 .4147822E-02 .4157750E-02 .4003017E-02 .4237310E-02 .4858845E-02 .5548946E-02 .5436018E-02 .4672021E-02 .3977059E-02 .3714260E-02 .3711222E-02 .3467249E-02 .3013643E-02 .2556512E-02 .2002048E-02 .1504113E-02 .1156737E-02 .6068208E-03 -.2876132E-03 -.1287015E-02 -.2522327E-02 -.3608660E-02 -.4053804E-02 -.3695964E-02 -.2949632E-02 -.2120182E-02 -.1552870E-02 -.1393597E-02 -.1484658E-02 -.1779394E-02 -.2539844E-02 -.3031609E-02 -.2564878E-02 -.1275312E-02 .4106712E-03 .2228657E-02 .3706974E-02 .5070756E-02 .6527148E-02 .7922758E-02 .8859406E-02 .9023617E-02 .8166272E-02 .6828870E-02 .5170112E-02 .3161940E-02 .1188943E-02 -.5270326E-03 -.2175452E-02 -.3523437E-02 -.4444128E-02 -.4641378E-02 -.4341658E-02 -.3851973E-02 -.2848926E-02 -.1390378E-02 .7927636E-04 .1011984E-02 .1275528E-02 .1070309E-02 .5956250E-03 .2962814E-03 -.2184010E-03 -.8908706E-03 -.1349708E-02 -.1493275E-02 -.1632731E-02 -.2403695E-02 -.3463080E-02 -.4261991E-02 -.4960756E-02 -.5779601E-02 -.6330778E-02 -.6060921E-02 -.5116714E-02 -.4120095E-02 -.3178005E-02 -.2288646E-02 -.1713921E-02 -.1522127E-02 -.1547654E-02 -.1746453E-02 -.2363664E-02 -.3290565E-02 -.4292185E-02 -.5504165E-02 -.6604991E-02 -.7624830E-02 -.8607745E-02 -.9274358E-02 -.9525798E-02 -.9518159E-02 -.9446399E-02 -.9268485E-02 -.8676048E-02 -.7728802E-02 -.6825317E-02 -.5928332E-02 -.5015031E-02 -.4376109E-02 -.4151987E-02 -.4180642E-02 -.4238801E-02 -.4234023E-02 -.4016659E-02 -.3328619E-02 -.2326878E-02 -.1274753E-02 .1079409E-03 .1690456E-02 .3225204E-02 .4613884E-02 .5773529E-02 .6455672E-02 .6802746E-02 .7233331E-02 .7823396E-02 .8574598E-02 .8886991E-02 .8206532E-02 .7193635E-02 .6571205E-02 .6350934E-02 .5947333E-02 .5145980E-02 .4222257E-02 .3338800E-02 .2601747E-02 .1941247E-02 .1523494E-02 .1381742E-02 .1394071E-02 .1425348E-02 .1413897E-02 .1369575E-02 .1328255E-02 .1313278E-02 .1337057E-02 .1392013E-02 .1429036E-02 .1238707E-02 .9043874E-03 .7234199E-03 .6740065E-03 .9547423E-03 .1316339E-02 .1516975E-02 .1524966E-02 .1429477E-02 .1342859E-02 .1308068E-02 .9224072E-03 -.5339943E-04 -.9859118E-03 -.1444202E-02 -.1459364E-02 -.1301286E-02 -.1199880E-02 -.1223197E-02 -.1296396E-02 -.9456147E-03 .2910030E-03 .2027471E-02 .3901765E-02 .5602304E-02 .6557772E-02 .6734670E-02 .6159994E-02 .4986350E-02 .3658287E-02 .2338046E-02 .1315313E-02 .3792467E-03 -.5084933E-03 .1090170E-02 -.1307222E-02 -.1307500E-02 -.1265381E-02 -.1269029E-02 -.1315794E-02 -.1364833E-02 -.1386445E-02 -.1379322E-02 -.1341620E-02 -.9177955E-03 -.8385196E-04 .3814120E-03 .5393167E-03 .3973293E-03 .3491824E-03 -.1150648E-02 .1927541E-02 -.2897546E-02 -.3665953E-02 -.4389704E-02 -.5408880E-02 .6272437E-02 -.6673335E-02 .6693899E-02 .6581844E-02 .6521480E-02 .6555714E-02 -.6633002E-02 .6689741E-02 .6681510E-02 .6240446E-02 .5180710E-02 -.4200898E-02 .3565643E-02 .2910376E-02 .2133490E-02 .1526218E-02 -.1062194E-02 .5404348E-03 .7660266E-04 .7805304E-03 .1270762E-02 .1441537E-02 .1411565E-02 .1553333E-02 .2063296E-02 .2771557E-02 .339569E-02 .3529763E-02 .3697577E-02 .3864150E-02 .3993231E-02 .4085136E-02 .4135812E-02 .4152865E-02 .4152730E-02 .4158613E-02 .4370192E-02 .5087897E-02 .6088067E-02 .6792159E-02 .6978185E-02 .6448985E-02 .5266110E-02 .4260411E-02 .3684008E-02 .3267152E-02 .2809849E-02 .2192877E-02 .1691521E-02 .1663826E-02 .2350588E-02 .3202394E-02 .3623873E-02 .3843732E-02 .3757466E-02 .3102015E-02 .2140866E-02 .1473825E-02 .1303137E-02 .1435291E-02 .1591219E-02 .1624270E-02 .1539462E-02 .1240541E-02 .6832456E-03 .3567133E-04 -.4402248E-03 .5602329E-03 .7029389E-03 .8996042E-03 .1060009E-02 -.1140377E-02 .1154306E-02 .1125554E-02 .1076118E-02 .1041646E-02 -.1034401E-02 .1043458E-02 .1034983E-02 .6335306E-03 .3657534E-03 .1294124E-02 .1708847E-02 .1675165E-02 .1500064E-02 .1604048E-02 .1955474E-02 .2016709E-02 .1820489E-02 .1599392E-02 .1482386E-02	-.5258219E-02 -.4910168E-02 .1764621E-02 .2268278E-02 .4031934E-02 -.7747279E-03 .1141628E-01 .2197989E-01 .1360154E-01 .1310647E-02 .1882040E-01 .1621936E-01 .1235463E-01 .1363416E-01 .1047111E-01 .4137391E-02 .5637855E-02 .6277400E-02 .1289048E-01 .2393038E-02 .2401986E-01 .1999760E-01 .8696973E-02 .8790182E-03 .2704388E-01 .6833355E-02 .8140883E-02 .4524632E-02 .1098149E-01 .5561641E-02 .1201174E-01 .1978921E-01 .1214361E-01 .6195855E-02 .6036099E-02 -.9787206E-03 .2674193E-02 .7506641E-02 .1351237E-01 .9938153E-02 .2914173E-02 .3620269E-02 .4911104E-02 .9064874E-03 .4855352E-03 .5367606E-02 .3078933E-02 .1519301E-02 .2663780E-02 .3306814E-03 -.1706595E-02 .4542932E-02 .6584969E-02 .1978166E-02 .1352218E-02 .9081856E-02 .9764238E-02 .4240942E-03 .1229686E-02 .3282072E-02 -.6313533E-02 .3084524E-02 .1738051E-02 .2183058E-02 .1253642E-02 .1010208E-02 .3335991E-02 .2285904E-02 .5173998E-02 .3045091E-02 -.3136502E-02 .5445488E-03 .2080593E-02 .1956864E-02 .4891318E-03 .2221212E-02 .7765332E-03 .1323953E-02 .2225822E-02 .1861494E-03 .7818464E-03 .1915377E-02 .2054785E-02 .3953936E-02 .1611736E-02 .3721611E-03 .1241485E-02 .2623116E-02 .1814024E-02 .1831143E-02 .1634738E-02 .5214185E-03 .4939520E-03 .9198790E-03 .9939785E-03 .1176840E-02 .1772992E-02 .4729283E-03 .1917341E-03 .5952489E-03 .1374931E-03 .2023874E-02 .2165519E-02 .2238449E-02 .1812144E-02 .1467562E-02 .1069803E-02 .5271167E-04 .3082822E-03 .6937768E-03 .3271820E-03 .5214269E-03 .4855708E-03 .2107220E-03 .7725041E-03 -.6112651E-03 .4363986E-03 .7416658E-03 .8904903E-03 .1018183E-02 -.9308720E-03 .6812777E-03 .6523080E-03 .9892000E-03 .1316070E-02 -.1256847E-02 .1248909E-02 .1312344E-02 .1328984E-02 .1213627E-02 -.1153174E-02 .1166860E-02 .1149369E-02 .1050604E-02 .8960229E-03 -.8367952E-03 .7659158E-03 .6844227E-03 .6588307E-03 .6126894E-03 -.5392488E-03 .4882042E-03 .4369489E-03 .3673823E-03 .3060702E-03 -.2599064E-03

APPENDIX I: PEER Ground Motion Data Base RSN454_MORGAN_GIL337.AT2	APPENDIX II: PEER Ground Motion Data Base RSN1633_MANJIL_ABBAR--L.AT2
.1264156E-02 .5902964E-03 .1735713E-03 .5654842E-03 .6324896E-03 .2854850E-03 .3029609E-03 .7785517E-03 .8798939E-03 .8041126E-03 .6636149E-03 .6640020E-03 .7231306E-03 .9985490E-03 .9073625E-03 .4535337E-04 .9093144E-03 .1405583E-02 .1420795E-02 .1230078E-02 .1091542E-02 .1085711E-02 .9759893E-03 .5571250E-03 .8940431E-04 .8503022E-03 .1367270E-02 .1536100E-02 .1506813E-02 .1448945E-02 .1448026E-02 .1499341E-02 .1557278E-02 .1578278E-02 .1388404E-02 .9000846E-03 .2415972E-03 .4728495E-03 .9111164E-03 .1006081E-02 .9403570E-03 .1278045E-02 .2311654E-02 .3329126E-02 .3802504E-02 .3755087E-02 .3519586E-02 .3374200E-02 .3393643E-02 .3501190E-02 .3578025E-02 .3565441E-02 .3501883E-02 .3445237E-02 .3423333E-02 .3416702E-02 .3396768E-02 .3372537E-02 .3355961E-02 .3335381E-02 .3298348E-02 .3265044E-02 .3258649E-02 .3451497E-02 .3752281E-02 .3704205E-02 .3409685E-02 .3130029E-02 .2618330E-02 .1626856E-02 .7446199E-03 .3547446E-03 .3529791E-03 .2454196E-03 .4323484E-03 .1459324E-02 .2237406E-02 .2489045E-02 .1996814E-02 .8362574E-03 .1696589E-03 .9477021E-03 .1854026E-02 .2569127E-02 .2890519E-02 .2918608E-02 .2843569E-02 .2789970E-02 .2783239E-02 .2413475E-02 .1419901E-02 .4503457E-03 .4132674E-03 .1432777E-02 .2211614E-02 .2531537E-02 .2523926E-02 .2431145E-02 .2404776E-02 .2441391E-02 .2100499E-02 .1113129E-02 .1546139E-03 .3178084E-03 .7266439E-03 .1578445E-02 .2425346E-02 .2877930E-02 .2940869E-02 .2832380E-02 .2747215E-02 .2762164E-02 .3060665E-02 .3349098E-02 .3251621E-02 .2956037E-02 .2726920E-02 .2664498E-02 .2718416E-02 .2791542E-02 .2822609E-02 .2807464E-02 .2774815E-02 .2752635E-02 .2750396E-02 .2709375E-02 .2154730E-02 .1320583E-02 .6812017E-03 .2416245E-03 .4641772E-04 .2674998E-04 .5736739E-04 .3179987E-04 .3268260E-03 .1204573E-02 .2214620E-02 .2834031E-02 .2957962E-02 .2818057E-02 .2684047E-02 .2644080E-02 .2196351E-02 .1249978E-02 .4396322E-03 .1270675E-03 .2126036E-03 .3996528E-03 .4844871E-03 .4393305E-03 .3426568E-03 .2760477E-03 .2698627E-03 .3192202E-03 .3723000E-03 .3499028E-03 .2069203E-03 .1272554E-02 .2138138E-02 .2480312E-02 .2689477E-02 .3292468E-02 .4145443E-02 .4819079E-02 .5605063E-02 .6614442E-02 .7312323E-02 .7017503E-02 .5902582E-02 .4923539E-02 .4572964E-02 .4717963E-02 .4933691E-02 .4486442E-02 .3360769E-02 .2315355E-02 .1332937E-02 .3359264E-03 .3061518E-03 .5027342E-03 .4496360E-03 .3755063E-03 .3855539E-03 .4603959E-03 .5341873E-03 .5640585E-03 .5509520E-03 .5635804E-03 .8759541E-03 .1072042E-02 .9067013E-03 .6238102E-03 .4887765E-03 .8815629E-03 .1867486E-02 .2860493E-02 .3434213E-02 .4159798E-02 .5074823E-02 .5723111E-02 .5973760E-02 .5948799E-02 .5847801E-02 .5808324E-02 .5850387E-02 .5903414E-02 .5594575E-02 .4716013E-02 .3755919E-02 .3229030E-02 .3174969E-02 .3256107E-02 .2659654E-02 .1549908E-02 .6911583E-03 .3985840E-03 .5079048E-03 .7034205E-03 .8257969E-03 .1185438E-02 .2003848E-02 .2868006E-02 .3259468E-02 .2595818E-02 .1363272E-02 .4965644E-03 .2197079E-03 .2839382E-03 .1141303E-02 .1842060E-02 .2142875E-02 .2094360E-02 .1578873E-02 .6072834E-03 .2952454E-03 .7104687E-03 .6923387E-03 .5256360E-03 .4353109E-03 .4744145E-03 .5751657E-03 .6523773E-03 .6694870E-03 .6034611E-03 .2205649E-03 .3716519E-03 .1114489E-02 .1772271E-02 .2067080E-02 .2045513E-02 .1924855E-02 .1866076E-02 .1900095E-02 .1934607E-02 .1596962E-02 .6975603E-03 .2694125E-03 .7944292E-03 .8844100E-03 .1112093E-02 .1895587E-02 .2885256E-02 .3863546E-02 .4895334E-02 .5712458E-02 .6079444E-02 .6097292E-02 .6006364E-02 .5969733E-02 .5994766E-02 .5957411E-02 .5285322E-02 .4175060E-02 .2996510E-02 .1829316E-02 .9833587E-03 .6494550E-03 .6661520E-03 .7632637E-03 .7797315E-03 .7072706E-03 .6159849E-03 .5665195E-03 .4896530E-03 .2012825E-03 .1279787E-02 .2042090E-02 .2259532E-02 .2156290E-02 .2022706E-02 .1980760E-02 .2046094E-02 .2154068E-02 .2221324E-02 .2201688E-02 .2129312E-02 .2068365E-02 .2009601E-02 .1631880E-02 .7442312E-03 .1877641E-03 .6634278E-03 .6707355E-03 .4903199E-03 .3702340E-03 .3865613E-03 .4807349E-03 .5664185E-03 .6964853E-03 .1410972E-02 .2469864E-02 .3202861E-02 .3505910E-02 .4136404E-02 .5086090E-02 .5813207E-02 .6102913E-02 .6075115E-02 .5919802E-02 .5459341E-02	

APPENDIX I: PEER Ground Motion Data Base RSN454_MORGAN_GIL337.AT2	APPENDIX II: PEER Ground Motion Data Base RSN1633_MANJIL_ABBAR--L.AT2
4578852E-02 .3719785E-02 .3289705E-02 .3226523E-02 .2951238E-02 .2115995E-02 .1157100E-02 .5922671E-03 .4567202E-03 .2091149E-03 -.5659261E-03 -.1496143E-02 -.2068983E-02 -.2189497E-02 -.2024235E-02 -.1483692E-02 -.5426815E-03 .3265975E-03 .7549063E-03 .7866217E-03 .6651243E-03 .5935642E-03 .6260268E-03 .6503228E-03 .3326660E-03 -.1709427E-03 -.2234321E-03 .1845614E-03 .6147362E-03 .8046074E-03 .7677558E-03 .6516482E-03 .5803665E-03 .5847657E-03 .6090047E-03 .6133427E-03 .5994600E-03 .5807226E-03 .5680225E-03 .5228067E-03 .1123911E-03 -.8075139E-03 -.1763771E-02 -.2282755E-02 -.2338213E-02 -.2185708E-02 -.2070135E-02 -.2083097E-02 -.2199337E-02 -.2322506E-02 -.2467761E-02 -.2954485E-02 -.3861874E-02 -.4708606E-02 -.5224475E-02 -.5762113E-02 -.6589683E-02 -.7378252E-02 -.7799386E-02 -.7863755E-02 -.7771483E-02 -.7706482E-02 -.7557173E-02 -.6788102E-02 -.5760360E-02 -.4915660E-02 -.3934199E-02 -.2851036E-02 -.1538992E-02 -.2112563E-03 .1197306E-02 .2328871E-02 .2855856E-02 .2893966E-02 .2744496E-02 .2651299E-02 .2502219E-02 .1763892E-02 .7759957E-03 .1364704E-03 -.1378951E-04 .1118509E-03 .2496395E-03 .2724525E-03 .2005011E-03 .1081673E-03 .3663127E-04 .7963861E-05 .2553487E-04 .2643221E-03 .1149181E-02 .2206390E-02 .2817860E-02 .2901535E-02 .2728483E-02 .2582887E-02 .2579221E-02 .2674061E-02 .2771540E-02 .2812048E-02 .2796435E-02 .2759957E-02 .2735412E-02 .2733770E-02 .2746918E-02 .2761094E-02 .2767986E-02 .2766937E-02 .2673724E-02 .2171640E-02 .1224498E-02 .3593261E-03 .3576909E-04 -.1841984E-04 .1414735E-03 .2314625E-03 .2001354E-03 .1096695E-03 .3955902E-04 .2411769E-04 -.4619690E-04 -.5365064E-03 -.1474774E-02 -.2328656E-02 -.2717793E-02 -.2796249E-02 -.3074502E-02 -.3649259E-02 -.4461048E-02 -.5119789E-02 -.5389418E-02 -.5357389E-02 -.5374825E-02 -.5637446E-02 -.5702576E-02 -.5536005E-02 -.5051020E-02 -.3996236E-02 -.2972697E-02 -.2472572E-02 -.2176317E-02 -.1400595E-02 -.4862723E-03 .8477145E-04 .2260761E-03 .1318194E-03 .2513836E-04 .1094693E-04 .7249214E-04 .1428151E-03 .3647543E-04 -.5808332E-03 -.1594595E-02 -.2404505E-02 -.2703878E-02 -.2622630E-02 -.2163019E-02 -.1137032E-02 -.1633281E-03 .2825659E-03 .4068338E-03 .8554433E-03 .1747408E-02 .2565380E-02 .2956533E-02 .2965872E-02 .2830271E-02 .2741489E-02 .2469966E-02 .1590396E-02 .6423791E-03 .1350335E-03 .1291485E-03 .7043648E-03 .1598727E-02 -.2314576E-02 -.2603268E-02 -.2569435E-02 -.2444403E-02 -.2376172E-02 -.2183771E-02 -.1517515E-02 -.5720152E-03 .1248401E-03 .7557226E-03 .1700175E-02 .2524748E-02 .2917755E-02 .2946141E-02 .2843548E-02 .2787699E-02 .2803267E-02 .2460584E-02 .1484757E-02 .5395162E-03 .9313287E-04 .1040040E-03 .2832707E-03 .3936524E-03 .3704147E-03 .2861737E-03 .4162242E-03 .7152744E-03 .7030866E-03 .4652576E-03 .2469814E-03 .1670033E-03 .2004637E-03 .2527868E-03 .1224907E-04 -.8196838E-03 -.1838854E-02 -.2470033E-02 -.2594655E-02 -.2464922E-02 -.2354027E-02 -.2344785E-02 -.2398744E-02 -.2475781E-02 -.2537903E-02 -.2506893E-02 -.1943395E-02 -.1149079E-02 -.1081881E-02 -.1736794E-02 -.2384509E-02 -.2175760E-02 -.1592589E-02 -.1335258E-02 -.1501155E-02 -.2018492E-02 -.2476499E-02 -.2649622E-02 -.2589416E-02 -.2463975E-02 -.2397668E-02 -.2414249E-02 -.2443900E-02 -.2164771E-02 -.1321985E-02 -.3012599E-03 .3578736E-03 .7896766E-03 .1459930E-02 .2290834E-02 .2877567E-02 .3132995E-02 .3663507E-02 .4645620E-02 .5462198E-02 .5506169E-02 .4652334E-02 .3541687E-02 .2884907E-02 .2863553E-02 .3659064E-02 .4921993E-02 .5808467E-02 .6023100E-02 .5771957E-02 .4968353E-02 .3841455E-02 .3078748E-02 .2849933E-02 .2443917E-02 .1554392E-02 .7206079E-03 .2589006E-03 .3379243E-03 .1279334E-02 -.2055540E-02 -.2334704E-02 -.1744535E-02 -.5503450E-03 .3737715E-03 .6550082E-03 .5395224E-03 .8550472E-03 .1824589E-02 .2777417E-02 .3245555E-02 .3234457E-02 .3021798E-02 .2890793E-02 .2929780E-02 .3004339E-02 .2533690E-02 .1491809E-02 .6003286E-03 .2445496E-03 .3117384E-03 .4796939E-03 .5328104E-03 .4353549E-03 .7359171E-04 -.2347555E-03 -.1397700E-03 .1578096E-03 .3875424E-03 .4478150E-03 .4147403E-03 .6406021E-03 .1421426E-02 .2410332E-02 .3047556E-02 .3195028E-02 .3039745E-02 .2579440E-02 .1727987E-02 .8407006E-03 .3489943E-03 .2919468E-03 .6531853E-03 .9767370E-03 .8422752E-03 .4770247E-03 .1983103E-03 .1277707E-03 .2026417E-03 .3010152E-03	

APPENDIX I: PEER Ground Motion Data Base RSN454_MORGAN_GIL337.AT2	APPENDIX II: PEER Ground Motion Data Base RSN1633_MANJIL_ABBAR--L.AT2
.3450526E-03 .3289645E-03 .2393980E-03 -.3329353E-03 -.1084972E-02 -.1105849E-02 -.4443370E-03 .2322193E-03 .5247383E-03 .4786465E-03 .3135251E-03 .2145370E-03 .2274364E-03 .2890888E-03 .3563316E-03 .3880041E-03 .4086012E-03 .6560217E-03 .9252488E-03 .1087509E-02 .1074321E-02 .7850474E-03 .4588597E-03 .2976761E-03 .3207726E-03 .3962402E-03 .4457213E-03 .7086697E-03 .1213788E-02 .1607224E-02 .1492434E-02 .9249103E-03 .3887885E-03 .1713347E-03 .2294387E-03 .3711557E-03 .4243767E-03 .1545598E-03 -.1752242E-03 -.4029437E-03 -.6675393E-03 -.8645912E-03 -.6896011E-03 -.1628594E-03 .2985065E-03 .4723589E-03 .4180211E-03 .3041788E-03 .2340989E-03 .2217222E-03 .2477362E-03 .2774644E-03 .2791966E-03 .5920774E-04 .6704710E-03 -.1676853E-02 -.2379472E-02 -.2578557E-02 -.2462922E-02 -.2310246E-02 -.2271725E-02 -.2340888E-02 -.2449137E-02 -.2537407E-02 -.2567387E-02 -.2549209E-02 -.2518283E-02 -.2501725E-02 -.2504677E-02 -.2516961E-02 -.2527165E-02 -.2544126E-02 -.2577738E-02 -.2607363E-02 -.2619613E-02 -.2617565E-02 -.2611130E-02 -.2607552E-02 -.2599170E-02 -.2390879E-02 -.1693267E-02 -.7340619E-03 -.6493203E-04 .1264684E-03 .2005610E-04 -.1212948E-03 -.1548666E-03 -.9737763E-04 -.2248033E-03 -.7201411E-03 -.1421765E-02 -.2170867E-02 -.2640108E-02 -.2754521E-02 -.2675251E-02 -.2604330E-02 -.2614964E-02 -.2494690E-02 -.1914689E-02 -.9558837E-03 -.1753699E-03 .1295077E-03 .7358233E-04 -.8694847E-04 -.1751953E-03 -.1276474E-03 .2602120E-03 .1260858E-02 .2261385E-02 .2464191E-02 .1657182E-02 .7865750E-03 .1605917E-03 -.1017851E-03 -.7792726E-04 -.1184338E-03 -.4222555E-03 -.6558151E-03 -.8326648E-03 -.6628396E-03 -.3170930E-03 -.8186834E-04 -.1694230E-04 .8068898E-04 .3478478E-03 .5049084E-03 .7553703E-03 .1178799E-02 .1812215E-02 .2057506E-02 .1331004E-02 .1400818E-03 -.1125060E-02 -.2194453E-02 -.2758061E-02 -.2847244E-02 -.2739228E-02 -.2673758E-02 -.2722192E-02 -.2827749E-02 -.2908594E-02 -.2927787E-02 -.2900727E-02 -.2864307E-02 -.2844980E-02 -.2846947E-02 -.2858137E-02 -.2729752E-02 -.2411671E-02 -.2374958E-02 -.2614241E-02 -.2890140E-02 -.3032917E-02 -.3030104E-02 -.2964043E-02 -.2915391E-02 -.2913296E-02 -.2941477E-02 -.2968764E-02 -.2838754E-02 -.2222159E-02 -.1221219E-02 -.4217618E-03 -.1237910E-03 -.1969039E-03 -.3693624E-03 -.4590403E-03 -.4192687E-03 -.3408850E-03 -.2864536E-03 -.2677343E-03 -.2979573E-03 -.3350420E-03 -.3400676E-03 -.3293782E-03 -.2993098E-03 -.2773364E-03 -.1478832E-03 .1611907E-03 .4589606E-03 .1018892E-02 .1479214E-02 .1917024E-02 .2252326E-02 .2404376E-02 .2430263E-02 .2438705E-02 .2468715E-02 .2359720E-02 .1756431E-02 .7760095E-03 -.2585934E-05 -.2908278E-03 -.2075311E-03 .1237005E-03 .6846320E-03 .1296054E-02 .1960088E-02 .2441022E-02 .2625484E-02 .2899807E-02 .3809338E-02 .4711734E-02 .4688080E-02 .3779478E-02 .2849011E-02 .2445742E-02 .2528345E-02 .2774833E-02 .2931590E-02 .2943884E-02 .2719038E-02 .2008797E-02 .9910270E-03 .2378865E-03 -.4756268E-05 .9578594E-04 .2550776E-03 .4514366E-03 .7400590E-03 .7123757E-03 .4408870E-03 .1991071E-03 .1199799E-03 .1694987E-03 .2589404E-03 .3321062E-03 .3568369E-03 .3412444E-03 .3158460E-03 .3142629E-03 .3529427E-03 .2523728E-03 -.2132074E-03 -.5914833E-03 -.3866353E-03 .1168898E-03 .4893127E-03 .5794703E-03 .4838767E-03 .3665489E-03 .3237624E-03 .3558092E-03 .4224618E-03 .4818940E-03 .4853907E-03 .4590574E-03 .4569193E-03 .4794662E-03 .5045426E-03 .5177793E-03 .5182521E-03 .5126622E-03 .5077588E-03 .5063312E-03 .5074614E-03 .5089044E-03 .5091510E-03 .5081020E-03 .5065008E-03 .5051154E-03 .5042305E-03 .5036946E-03 .5031940E-03 .5025372E-03 .5016991E-03 .5007850E-03 .4998893E-03 .4990549E-03 .4982588E-03 .4974633E-03 .4966406E-03 .4957948E-03 .4949299E-03 .4940633E-03 .4931981E-03 .4923398E-03 .4914732E-03 .4906025E-03 .4897211E-03 .4888407E-03 .4879500E-03 .4870664E-03 .4765058E-03 .2681666E-03 -.5183256E-04 -.2947247E-04 .2063697E-03 .4095590E-03 .4643074E-03 .2194744E-03 -.3445276E-03 -.8383590E-03 -.1044580E-02 -.1207957E-02 -.1414874E-02 -.1313291E-02 -.8625011E-03 -.1861793E-03 .3108902E-03 .4643117E-03 .3909974E-03 .2835913E-03 .2528965E-03 .2984912E-03 .3734531E-03 .6274783E-03 .1151745E-02 .1422666E-02 .1110833E-02 .6013648E-03 .3041467E-03 .2931594E-03 .4236570E-03 .5327892E-03 .5511054E-03 .5022338E-03 .4532960E-03 .6189640E-03 .9361339E-03	

APPENDIX I: PEER Ground Motion Data Base RSN454_MORGAN_GIL337.AT2	APPENDIX II: PEER Ground Motion Data Base RSN1633_MANJIL_ABBAR--L.AT2
.9588966E-03 .7542449E-03 .5497583E-03 .4666311E-03 .4818972E-03 .3238717E-03 -.1679020E-03 -.4527190E-03 -.1647815E-03 .3178360E-03 .4046761E-03 -.2903373E-03 -.1392471E-02 -.2179655E-02 -.2393750E-02 -.2254045E-02 -.2078050E-02 -.2015487E-02 -.2071029E-02 -.2166059E-02 -.2226104E-02 -.2229036E-02 -.2197802E-02 -.2166906E-02 -.2155782E-02 -.2163164E-02 -.2176884E-02 -.2186248E-02 -.2187995E-02 -.2185110E-02 -.2173129E-02 -.1961307E-02 -.1248774E-02 -.2546665E-03 .4451382E-03 .6342824E-03 .5017865E-03 .3547944E-03 .3355057E-03 .4281651E-03 .7268863E-03 .1262010E-02 .1521448E-02 .1401837E-02 .1223655E-02 .1097592E-02 .1341978E-02 .1925920E-02 .2501023E-02 .2725215E-02 .2904288E-02 .2868059E-02 .2452335E-02 .2012894E-02 .1624173E-02 .1143111E-02 .7645158E-03 .6032071E-03 .5922166E-03 .4328214E-03 -.4746346E-04 -.7146306E-03 -.1443779E-02 -.1893047E-02 -.1990253E-02 -.1901194E-02 -.1813922E-02 -.1820434E-02 -.2073885E-02 -.2433680E-02 -.2425731E-02 -.2161517E-02 -.1915567E-02 -.1823645E-02 -.1846799E-02 -.1884290E-02 -.1891032E-02 -.1869155E-02 -.1830614E-02 -.1586883E-02 -.8576987E-03 -.8799037E-04 .5679098E-04 -.4251041E-03 -.1044203E-02 -.1106784E-02 -.5641587E-03 .2959278E-03 .7178265E-03 .1730961E-03 -.9393829E-03 -.1790461E-02 -.2037687E-02 -.1867889E-02 -.1630760E-02 -.1541649E-02 -.1610310E-02 -.1732576E-02 -.1811407E-02 -.1817660E-02 -.1779854E-02 -.1741404E-02 -.1727037E-02 -.1735579E-02 -.1752092E-02 -.1763297E-02 -.1717200E-02 -.1161770E-02 -.1063623E-03 .7651602E-03 .1081887E-02 .9877531E-03 .8111520E-03 .7313363E-03 .7783539E-03 .8916484E-03 .9780082E-03 .9967111E-03 .9667055E-03 .9288803E-03 .9106736E-03 .9387638E-03 .1253164E-02 .2074287E-02 .3046547E-02 .3642871E-02 .3759477E-02 .3624197E-02 .3513428E-02 .3541627E-02 .3653311E-02 .3728114E-02 .3488321E-02 .2953452E-02 .2476297E-02 .2025632E-02 .1551998E-02 .1235908E-02 .1130664E-02 .1149750E-02 .1160968E-02 .8582866E-03 .1562371E-04 .9692661E-03 -.1546590E-02 -.1632529E-02 -.1474047E-02 -.1336278E-02 -.1327355E-02 -.1392151E-02 -.1442757E-02 -.1448826E-02 -.1398671E-02 -.1090661E-02 -.5371742E-03 -.8107580E-04 .7103695E-04 .2425409E-03 -.8521999E-03 -.1310155E-02 -.1436198E-02 -.1338380E-02 -.1168072E-02 -.8127319E-03 -.2833252E-03 .3721554E-03 .1034881E-02 .1433018E-02 .1528440E-02 .1469539E-02 .1388159E-02 .1121550E-02 .6598884E-03 .2940560E-03 .1598416E-03 .1932372E-03 .2820811E-03 .3445983E-03 .3066806E-03 .1000866E-03 -.7039397E-03 -.1108538E-02 -.1219875E-02 -.1503462E-02 -.2256626E-02 -.2768362E-02 -.2390116E-02 -.1537693E-02 -.9109493E-03 -.7555080E-03 -.9069024E-03 -.1092614E-02 -.1155486E-02 -.1097610E-02 -.1003339E-02 -.9461835E-03 .9453381E-03 .9766535E-03 .1006511E-02 .1016987E-02 -.1010121E-02 -.9978588E-03 .9902803E-03 .9511465E-03 .6043104E-03 -.9856127E-05 .7488762E-03 .1452776E-02 .2137053E-02 .2697072E-02 .2944178E-02 .2638460E-02 .2043153E-02 .1632262E-02 .1552796E-02 .1670901E-02 .1794304E-02 .1823809E-02 .1772421E-02 .1703919E-02 .1667513E-02 .1671742E-02 .1696255E-02 .1717030E-02 .1723001E-02 .1678239E-02 .1321749E-02 .7425731E-03 .3929074E-04 .2980302E-03 -.3859193E-03 -.2059966E-03 .2091869E-03 .8307972E-03 .1425189E-02 .1724736E-02 .1744486E-02 .1655920E-02 .1645367E-02 .2043715E-02 .2676744E-02 .3099023E-02 .3178040E-02 .3060758E-02 .2900691E-02 .2523120E-02 .2038401E-02 .1731350E-02 .1665449E-02 .1687572E-02 .1376080E-02 .7671627E-03 .2169183E-04 .7058187E-03 .1022577E-02 -.1022924E-02 .9132787E-03 .8542422E-03 .8448339E-03 .6067215E-03 -.4751479E-03 .6472611E-03 .8595469E-03 .6929707E-03 .5128322E-03 -.5705309E-03 .4585803E-03 .4201223E-03 .6352266E-03 .8907019E-03 -.1024028E-02 .1024618E-02 .9677652E-03 .9257877E-03 .9243325E-03 .9488056E-03 .9729535E-03 .9815392E-03 .9755202E-03 .9645268E-03 .9570514E-03 .9556661E-03 .9579988E-03 .9604066E-03 .9607886E-03 .9592648E-03 .9570563E-03 .9552431E-03 .9542202E-03 .9536674E-03 .9531237E-03 .9523185E-03 .9512545E-03 .9500984E-03 .9489813E-03 .9479587E-03 .9469871E-03 .9460102E-03 .9449850E-03 .9439183E-03 .9428226E-03 .9417733E-03 .9018521E-03 .5110180E-03 .3973734E-03 .1343348E-02 .1844118E-02 .1884791E-02 .1727084E-02 .1613827E-02 .1627836E-02 .1680098E-02 .1415370E-02 .9114827E-03 .5214398E-03 .3884715E-03 .4367042E-03 .4811801E-03 .1411937E-03 .4571436E-03	

APPENDIX I: PEER Ground Motion Data Base RSN454_MORGAN_GIL337.AT2	APPENDIX II: PEER Ground Motion Data Base RSN1633_MANJIL_ABBAR--L.AT2
-9023308E-03 -1038761E-02 -9664734E-03 -8580817E-03 -8176495E-03 -8499149E-03 -9058505E-03 -9403989E-03 -9407122E-03 -9207570E-03 -9010006E-03 -8928051E-03 -8953176E-03 -9013857E-03 -9047858E-03 -9037018E-03 -9005394E-03 -9929397E-03 -1425817E-02 -1977472E-02 -2316983E-02 -2379073E-02 -2293767E-02 -2211048E-02 -2103789E-02 -1712075E-02 -1208696E-02 -8897813E-03 -7293979E-03 -3878480E-03 .1501411E-03 .8648790E-03 .1477549E-02 .1767748E-02 .1804055E-02 .1752577E-02 .1728874E-02 .1760876E-02 .1817381E-02 .1858764E-02 .1868915E-02 .1856862E-02 .1841033E-02 .1833949E-02 .1837154E-02 .1845096E-02 .1851736E-02 .1854520E-02 .1854453E-02 .1853251E-02 .1757545E-02 .1324234E-02 .7736036E-03 .3466445E-03 .1185315E-03 -5462860E-03 -7802283E-03 -8277828E-03 -7932007E-03 -7673978E-03 -7799567E-03 -8139255E-03 -9355409E-03 -1279900E-02 -1386124E-02 -1070060E-02 -3804117E-03 -2806868E-03 -5903446E-03 -5758284E-03 .4437598E-03 .2777156E-03 -1471646E-04 -4950283E-04 -2092033E-03 .4842079E-03 .6045829E-03 .6731593E-03 .1029911E-02 .1535073E-02 .1880850E-02 .1979079E-02 .1926874E-02 .1857714E-02 .1842310E-02 .1876539E-02 .1921604E-02 .1947189E-02 .2035328E-02 .2335975E-02 .2337667E-02 .1702816E-02 .8094064E-03 -6348526E-04 -6312409E-03 -7954157E-03 -7230415E-03 -6304457E-03 -6257361E-03 -7891397E-03 -1298156E-02 -1884079E-02 -2213408E-02 -2246447E-02 -2139242E-02 -2052491E-02 -2048967E-02 -2101775E-02 -2155179E-02 -2175247E-02 -2162978E-02 -2138354E-02 -2025275E-02 -1586450E-02 -1038654E-02 -7039413E-03 -6420554E-03 -7227207E-03 -7982453E-03 -8036046E-03 -7563213E-03 -7045814E-03 -6805318E-03 -6856871E-03 -7021910E-03 -6179197E-03 -1831439E-03 .3769113E-03 .7185255E-03 .7553290E-03 .6331352E-03 .5280629E-03 .5144760E-03 .5672162E-03 .6263042E-03 .6528229E-03 .6446925E-03 .5343881E-03 .1160144E-03 -3975006E-03 -7043402E-03 -7529995E-03 -6702919E-03 -6910410E-03 -1118528E-02 -1713049E-02 -2098345E-02 -2179709E-02 -2086530E-02 -1985840E-02 -1961381E-02 -2002351E-02 -2056356E-02 -2083543E-02 -2077560E-02 -2054369E-02 -1909731E-02 -1444473E-02 -9084272E-03 -6014578E-03 -5592153E-03 -6444998E-03 -8103742E-03 -1244294E-02 -1746215E-02 -1910320E-02 -1493363E-02 -8772617E-03 -5018115E-03 -4593003E-03 -5908901E-03 -7067242E-03 -5845572E-03 -6588640E-04 -4964540E-03 .7903693E-03 .8017614E-03 .6910388E-03 .6103448E-03 .6132998E-03 .6682561E-03 .6055824E-03 .2007397E-03 -3092238E-03 -6185609E-03 -6713341E-03 -5888446E-03 -4211775E-03 .9061199E-04 .9997830E-03 .1823545E-02 .2202710E-02 .2195834E-02 .2057001E-02 .1983167E-02 .2020129E-02 .2109105E-02 .2176949E-02 .2192150E-02 .2073973E-02 .1610423E-02 .1044700E-02 .7094393E-03 .6594653E-03 .7539963E-03 .8388591E-03 .9426060E-03 .1235082E-02 .1397982E-02 .1599864E-02 .1885483E-02 .2100983E-02 .2183907E-02 .2174631E-02 .2055186E-02 .1642341E-02 .1144870E-02 .7567051E-03 .2780633E-03 -1946721E-03 -4654981E-03 -5248724E-03 -4828111E-03 -3498457E-03 .7796657E-04 .5953552E-03 .9044958E-03 .9585824E-03 .8848143E-03 .8195067E-03 .7241945E-03 .3360037E-03 -1686975E-03 -4882005E-03 -5575603E-03 -4901494E-03 -4205833E-03 -4110978E-03 -4488747E-03 -4917156E-03 -5104264E-03 -5023608E-03 -3943741E-03 .2475852E-04 .6382281E-03 .1386039E-02 .1992911E-02 .2253913E-02 .2244577E-02 .2155304E-02 .2023909E-02 .1627892E-02 .1053458E-02 .3546550E-03 -2149488E-03 -4650628E-03 -4629395E-03 -3831059E-03 -3477853E-03 -2842233E-03 -2071006E-05 .1730258E-03 .2963878E-03 .1560890E-03 -1823518E-03 -4460478E-03 -5232846E-03 -4712684E-03 -3983514E-03 -3708133E-03 -3912197E-03 -4273309E-03 -4495144E-03 -4492363E-03 -4355721E-03 -4213538E-03 -3191089E-03 .1150753E-03 .6642879E-03 .1002592E-02 .1066742E-02 .9858131E-03 .9016268E-03 .8627214E-03 .7800861E-03 .4031295E-03 -9044869E-04 -4049387E-03 -3865494E-03 -5976511E-05 .1732411E-03 -1227270E-04 -3050982E-03 -4835092E-03 -4990620E-03 -3375428E-03 .1346010E-03 .6667441E-03 .9607965E-03 .9884187E-03 .8945121E-03 .8229975E-03 .8270503E-03 .8804522E-03 .1027017E-02 .1385714E-02 .1493872E-02 .1262921E-02 .1070182E-02 .1358829E-02 .1922577E-02 .2341328E-02 .2462306E-02 .2383882E-02 .2274506E-02 .2234326E-02 .2171715E-02 .1794088E-02 .1278154E-02 .9410508E-03	

APPENDIX I: PEER Ground Motion Data Base RSN454_MORGAN_GIL337.AT2	APPENDIX II: PEER Ground Motion Data Base RSN1633_MANJIL_ABBAR--L.AT2
.8609763E-03 .9278234E-03 .1002503E-02 .1016700E-02 .9800650E-03 .9352944E-03 .8748017E-03 .5712565E-03 .4044042E-03 .5798436E-03 .7805030E-03 .5655391E-03 .6121068E-04 .3404711E-03 .4733732E-03 .4112994E-03 .3076489E-03 .2623781E-03 .2859064E-03 .3778599E-03 .7865253E-03 .1363478E-02 .1746629E-02 .1874980E-02 .2141417E-02 .2583743E-02 .2936860E-02 .3080051E-02 .3066143E-02 .3007485E-02 .2979836E-02 .2995722E-02 .3029149E-02 .3051985E-02 .3054207E-02 .3042894E-02 .3030529E-02 .2985853E-02 .2638939E-02 .2109947E-02 .1741133E-02 .1640968E-02 .1748740E-02 .2206783E-02 .2801214E-02 .3163790E-02 .3220763E-02 .3080687E-02 .2642238E-02 .2043579E-02 .1269671E-02 .5734612E-03 .2301622E-03 .2021853E-03 .2902559E-03 .1538755E-03 .6218304E-03 .1411428E-02 .1970699E-02 .2228903E-02 .2005607E-02 .1778561E-02 .1566633E-02 .1309887E-02 .1137658E-02 .1017002E-02 .7003791E-03 .5829633E-03 .1055184E-02 .1387074E-02 .9682177E-03 .1813437E-03 .3773854E-03 .5133974E-03 .3813429E-03 .2213438E-03 .1687010E-03 .2198594E-03 .3010943E-03 .3493155E-03 .3487976E-03 .3209836E-03 .2949253E-03 .2858339E-03 .2916381E-03 .3019036E-03 .3080117E-03 .3077686E-03 .3039098E-03 .3002358E-03 .2987541E-03 .2992234E-03 .3002910E-03 .3008385E-03 .3005614E-03 .2998090E-03 .2990899E-03 .2986568E-03 .2984841E-03 .2983979E-03 .2982969E-03 .2592002E-03 .8903457E-04 .5831298E-03 .6066494E-03 .2128990E-03 .8283880E-04 .8236040E-04 .3303070E-04 .1818743E-03 .2769155E-03 .3034192E-03 .2924705E-03 .2787446E-03 .2779778E-03 .2874690E-03 .2975592E-03 .3017845E-03 .2999837E-03 .2958145E-03 .2928294E-03 .2922986E-03 .2933590E-03 .3364643E-03 .7115742E-03 .1284923E-02 .1723079E-02 .2139310E-02 .2208896E-02 .1622509E-02 .1128296E-02 .1173908E-02 .1460993E-02 .1360357E-02 .8621554E-03 .3887081E-03 .1784518E-03 .2021362E-03 .3081204E-03 .3736032E-03 .3655186E-03 .3173825E-03 .2756978E-03 .3054751E-03 .6503199E-03 .8674867E-03 .7039583E-03 .4112525E-03 .2243552E-03 .1999256E-03 .2666399E-03 .3316519E-03 .3488738E-03 .3258034E-03 .2936550E-03 .2763464E-03 .2782158E-03 .2897637E-03 .2995921E-03 .3024038E-03 .2996279E-03 .2955301E-03 .2933332E-03 .2936521E-03 .2952567E-03 .2966691E-03 .2971764E-03 .2969532E-03 .2965613E-03 .2964215E-03 .2965714E-03 .3318174E-03 .6319224E-03 .9255367E-03 .1113973E-02 .1059013E-02 .7013883E-03 .3501321E-03 .2025173E-03 .2545798E-03 .3832501E-03 .4653648E-03 .4656065E-03 .4199016E-03 .3786803E-03 .3672937E-03 .3807154E-03 .4001923E-03 .4111747E-03 .4107350E-03 .4043161E-03 .3987052E-03 .3990292E-03 .4220241E-03 .4643565E-03 .4980196E-03 .5099468E-03 .5060778E-03 .4987489E-03 .4959472E-03 .4984592E-03 .5029110E-03 .4820107E-03 .2077846E-03 .3082539E-03 .7256098E-03 .9045486E-03 .1157681E-02 .1623002E-02 .2034110E-02 .2201163E-02 .1954788E-02 .1628633E-02 .1416544E-02 .1177961E-02 .9681470E-03 .6440005E-03 .6069121E-03 .6723788E-03 .3281795E-03 .8167026E-04 .3207778E-04 .1011145E-03 .2724572E-03 .3869808E-03 .3981263E-03 .8570878E-04 .7430054E-03 .1716815E-02 .2307503E-02 .2417145E-02 .2276667E-02 .2145765E-02 .2139317E-02 .2242732E-02 .2370269E-02 .2440152E-02 .2438188E-02 .2400634E-02 .2343377E-02 .2040589E-02 .1473889E-02 .7659747E-03 .9383284E-04 .2784643E-03 .3418629E-03 .2649129E-03 .2064828E-03 .2226247E-03 .2835008E-03 .3365043E-03 .3535386E-03 .3401255E-03 .3180977E-03 .3050137E-03 .3051887E-03 .3119037E-03 .3026060E-03 .2843691E-03 .2873857E-03 .3042506E-03 .3187709E-03 .3237973E-03 .3215752E-03 .3176871E-03 .3159735E-03 .3169423E-03 .3093970E-03 .8491489E-04 .6428578E-03 .1426019E-02 .1395081E-02 .5864124E-03 .2314145E-03 .5830572E-03 .5070620E-03 .2877327E-03 .1579632E-03 .1769928E-03 .2766204E-03 .3782136E-03 .4253771E-03 .3962597E-03 .3382740E-03 .3008720E-03 .3131397E-03 .3503040E-03 .3513043E-03 .8442004E-04 .6796032E-03 .1446570E-02 .1765219E-02 .1563432E-02 .6962167E-03 .1583836E-03 .5324312E-03 .4621596E-03 .5498483E-04 .5589880E-03 .1006229E-02 .1157245E-02 .1300377E-02 .1522191E-02 .1457996E-02 .1229579E-02 .1046477E-02 .8024805E-03 .3485739E-03 .6126254E-04 .2515204E-03 .2532159E-03 .1893807E-03 .1544752E-03 .1508755E-03 .5013206E-04 .5318805E-03 .1168149E-02 .1685585E-02 .1839568E-02 .1810896E-02	

APPENDIX I: PEER Ground Motion Data Base RSN454_MORGAN_GIL337.AT2	APPENDIX II: PEER Ground Motion Data Base RSN1633_MANJIL_ABBAR--L.AT2
.1696396E-02 .1681716E-02 .1313444E-02 .5656566E-03 .6805744E-04 -.3226591E-03 -.2808752E-03 -.1572911E-03 -.1010202E-03 -.1357573E-03 -.2072190E-03 -.2569345E-03 -.2637922E-03 -.2422239E-03 -.2179965E-03 -.2072466E-03 -.2106066E-03 -.2196523E-03 -.2261186E-03 -.2270065E-03 -.2241213E-03 -.2208268E-03 -.2192491E-03 -.2194999E-03 -.2204941E-03 -.2204679E-03 -.2055844E-03 -.1822793E-03 -.1844221E-03 -.2036597E-03 -.2205975E-03 -.2262507E-03 -.2229982E-03 -.2175537E-03 -.2147007E-03 -.2151959E-03 -.2171982E-03 -.2186524E-03 -.2187096E-03 -.2177625E-03 -.2181930E-03 -.2360655E-03 -.2744885E-03 -.3079282E-03 -.3215504E-03 -.2976246E-03 -.1913445E-04 .7591895E-03 .1737899E-02 .2370482E-02 .2518415E-02 .2392354E-02 .2230040E-02 .1914425E-02 .1459890E-02 .1328006E-02 .1172219E-02 .5950227E-03 -.2375294E-04 -.3770053E-03 -.4351894E-03 -.3527575E-03 -.2962043E-03 -.3139809E-03 -.3728934E-03 -.4228937E-03 -.4388438E-03 -.4266252E-03 -.4064602E-03 -.3943205E-03 -.3921355E-03 -.3771378E-03 -.3431208E-03 -.3125636E-03 -.2987034E-03 -.2988387E-03 -.3035642E-03 -.3058329E-03 -.3040720E-03 -.3004501E-03 -.2975684E-03 -.2965192E-03 -.2967903E-03 -.2972637E-03 -.2972068E-03 -.2707283E-03 .3896698E-06 .2599991E-03 .1543487E-03 -.1029635E-03 -.4249467E-04 .1671150E-03 .1174483E-03 -.9738845E-04 -.2817010E-03 -.3487735E-03 -.3243851E-03 -.2759801E-03 -.2511530E-03 -.2580003E-03 -.2791149E-03 -.2949059E-03 -.2974111E-03 -.2904156E-03 -.2819841E-03 -.2775283E-03 -.2775995E-03 -.2796830E-03 -.2811073E-03 -.2807952E-03 -.2792270E-03 -.2774441E-03 -.2761711E-03 -.2754730E-03 -.2750478E-03 -.2726600E-03 -.2765433E-03 -.5060877E-03 -.7575987E-03 -.9344919E-03 -.1231788E-02 -.1750673E-02 -.2384385E-02 -.2829223E-02 -.2989764E-02 -.2961873E-02 -.2894760E-02 -.2876998E-02 -.2912823E-02 -.2962358E-02 -.2990381E-02 -.2988855E-02 -.2971203E-02 -.2954898E-02 -.2948907E-02 -.2927923E-02 -.2632915E-02 -.1824914E-02 -.8531187E-03 -.2489399E-03 -.1237269E-03 -.2561754E-03 -.3875275E-03 -.4000297E-03 -.3207232E-03 -.2324930E-03 -.1680390E-03 .1200025E-03 .8775457E-03 .1785111E-02 .2350648E-02 .2471678E-02 .2353282E-02 .2234626E-02 .2224446E-02 .2298001E-02 .2379253E-02 .2416441E-02 .2406587E-02 .2377078E-02 .2331595E-02 .2053790E-02 .1524682E-02 .8451947E-03 .1832757E-03 -.1909134E-03 -.2581715E-03 -.1812188E-03 -.1189419E-03 -.1303116E-03 -.1879539E-03 -.2397605E-03 -.2566853E-03 -.2430370E-03 -.2200174E-03 -.2053473E-03 -.2037700E-03 -.2097876E-03 -.2156136E-03 -.2170106E-03 -.2143790E-03 -.2104735E-03 -.2076042E-03 -.2064185E-03 -.2062357E-03 -.2060368E-03 -.2052519E-03 -.2039528E-03 -.2024821E-03 -.2011496E-03 -.2000282E-03 -.1990653E-03 -.1742002E-03 .1273250E-03 .9374403E-03 .1890165E-02 .2472191E-02 .2558172E-02 .2155060E-02 .1767115E-02 .1822234E-02 .1864338E-02 .1364760E-02 .7774467E-03 .6674717E-03 .8964822E-03 .1171102E-02 .1547114E-02 .2041570E-02 .2411225E-02 .2545269E-02 .2519304E-02 .2462341E-02 .2450077E-02 .2484346E-02 .2529516E-02 .2529328E-02 .2233185E-02 .1640749E-02 .9054115E-03 .2724824E-03 .4275348E-03 .8757146E-03 .7566329E-03 .2701877E-03 -.1037807E-03 .3076777E-03 .1406259E-02 .2067355E-02 .1620283E-02 .5587699E-03 -.2535485E-03 -.4753907E-03 -.2911278E-03 -.4511026E-04 .5083035E-04 -.1226897E-04 .1306389E-03 -.2072406E-03 -.2122567E-03 -.1733300E-03 -.1333521E-03 -.1170129E-03 -.1233838E-03 -.1396237E-03 -.1681414E-03 -.1883649E-03 -.1941686E-03 -.1862807E-03 -.1624577E-03 -.1578357E-03 -.1681034E-03 -.1814225E-03 -.1845453E-03 -.1638742E-03 -.1378712E-03 -.1228227E-03 -.1208589E-03 -.1251324E-03 -.1781773E-03 -.7482969E-03 -.1814655E-02 -.2675739E-02 -.2991162E-02 -.2902692E-02 -.2718900E-02 -.2642735E-02 -.2697360E-02 -.2800157E-02 -.2868177E-02 -.2874191E-02 -.2841045E-02 -.2805933E-02 -.2790993E-02 -.2757120E-02 -.2418671E-02 -.1867543E-02 -.1416692E-02 -.9354328E-03 -.4577497E-03 -.1746253E-03 -.1033976E-03 .1372431E-03 -.1699475E-03 -.1598787E-03 -.1218840E-03 -.8781904E-04 .7573955E-04 .8291622E-04 .9596967E-04 -.1037441E-03 -.1032541E-03 .9796913E-04 .9281239E-04 .9043484E-04 .9065588E-04 .5287682E-04 .3364479E-03 .1247475E-02 .2220488E-02 .2759449E-02 .2804735E-02 .2636960E-02 .2506454E-02 .2516898E-02 .2607409E-02 .2701012E-02 .2753698E-02 .2754212E-02 .2727344E-02 .2704163E-02 .2699434E-02 .2709356E-02 .2722163E-02 .2729191E-02 .2729054E-02 .2725392E-02 .2722425E-02 .2722040E-02 .2723698E-02	

APPENDIX I: PEER Ground Motion Data Base RSN454_MORGAN_GIL337.AT2	APPENDIX II: PEER Ground Motion Data Base RSN1633_MANJIL_ABBAR--L.AT2
.2725808E-02 .2727282E-02 .2647018E-02 .1924400E-02 .8570187E-03 .4342411E-03 .8020368E-03 .1689830E-02 .2541795E-02 .2878273E-02 .2449689E-02 .1676299E-02 .1125060E-02 .7012200E-03 .3460218E-03 .1513420E-03 .9729646E-04 .9941031E-04 .9393416E-04 .6676773E-04 .3488096E-04 .1685412E-04 .3508498E-04 .5135679E-03 .1473512E-02 -.2332867E-02 -.2612075E-02 -.2077355E-02 -.9911269E-03 -.1714880E-03 -.3596464E-03 -.1430532E-02 -.2506943E-02 -.2982556E-02 -.2896928E-02 -.2616652E-02 -.2448236E-02 -.2472899E-02 -.2600983E-02 -.2622153E-02 -.2249712E-02 -.1608754E-02 -.8100471E-03 -.1719121E-03 .8887942E-04 .6081625E-04 .5307076E-04 .1270639E-03 .1248520E-03 .7713422E-04 -.1230860E-03 .5157188E-03 .9565667E-03 .9804847E-03 .9423558E-03 -.9722992E-03 .7435444E-03 .3514366E-03 .7222909E-04 .3259610E-05 -.4853524E-04 .1135807E-03 .1301441E-03 .1018694E-03 .6336437E-04 -.4247115E-04 .4452410E-04 .5836333E-04 .7039044E-04 .7411339E-04 -.7107985E-04 .6636869E-04 .6387606E-04 .6439384E-04 .6649768E-04 -.6840686E-04 .6925978E-04 .6924749E-04 .6902296E-04 .6909150E-04 -.6956473E-04 .7025824E-04 .7094188E-04 .7149787E-04 .7195641E-04 -.7240165E-04 .7289937E-04 .7345481E-04 .7405756E-04 .7467089E-04 -.7527924E-04 .7588968E-04 .7650974E-04 .7715531E-04 .7781822E-04 -.7849060E-04 .7918064E-04 .7987918E-04 .8059026E-04 .8131088E-04 -.8195555E-04 .7450651E-04 .3410497E-03 .1295320E-02 .2327207E-02 -.2710635E-02 .2116615E-02 .9593045E-03 .6777590E-04 .2053101E-03 .5543284E-04 .1866479E-03 .2937166E-03 .2371391E-03 .1055219E-03 .2165757E-05 .2441274E-04 .2498334E-05 .2116361E-04 .2819562E-04 -.1953192E-04 .5962114E-05 .2694037E-05 .3360267E-05 .1158358E-05 -.6300464E-05 .9253163E-05 .9840421E-05 .9475308E-05 .9537164E-05 -.1291825E-04 .2943220E-03 .1227427E-02 .2240648E-02 .2764662E-02 -.2781663E-02 .2588476E-02 .2454556E-02 .2467677E-02 .2568342E-02 -.2660128E-02 .2691294E-02 .2669800E-02 .2632916E-02 .2611210E-02 -.2611170E-02 .2484183E-02 .1886687E-02 .9029416E-03 .1099106E-03 .1906454E-03 .1226904E-03 .2616404E-04 .5687609E-04 .4371710E-04 .1921520E-03 .2906074E-03 .3067178E-03 .2715661E-03 .2328042E-03 .2177593E-03 .2362560E-03 .2869271E-03 .3446106E-03 .3989083E-03 .4310565E-03 .4372088E-03 .4298870E-03 .4228952E-03 .4220406E-03 .4253993E-03 .4291736E-03 .4447614E-03 .4706357E-03 .2762658E-03 -.4481598E-03 .1445439E-02 .2136709E-02 .2329870E-02 .2215253E-02 -.2067312E-02 .2033593E-02 .2105281E-02 .2200046E-02 .2252078E-02 -.2249288E-02 .2218685E-02 .2183063E-02 .1969637E-02 .1642780E-02 -.1276868E-02 .5277707E-03 .1890310E-03 .5382111E-03 .5487988E-03 .4308707E-03 .3576318E-03 .3674880E-03 .2093848E-03 .4736778E-03 -.1470629E-02 .2212119E-02 .2463095E-02 .2379824E-02 .2224320E-02 -.1966472E-02 .1313113E-02 .4095352E-03 .2359231E-03 .4378504E-03 .3563197E-03 .2334795E-03 .2032420E-03 .2621730E-03 .3407240E-03 .3821712E-03 .3758810E-03 .3445299E-03 .3022359E-03 .2545161E-03 .2236304E-03 .2158727E-03 .2206415E-03 .2255985E-03 .2383021E-03 .2548547E-03 .2603903E-03 .2744414E-03 .2910122E-03 .3004370E-03 .3011152E-03 .2970420E-03 .2926510E-03 .2899973E-03 .2887149E-03 .2875467E-03 .2862933E-03 .2990549E-03 .3342320E-03 .3667733E-03 .3799744E-03 .3762750E-03 .3668437E-03 .3604221E-03 .3590799E-03 .3609210E-03 .3767656E-03 .4130815E-03 .4449157E-03 .4567787E-03 .4521662E-03 .4425883E-03 .4371763E-03 .4516886E-03 .4911906E-03 .5268712E-03 .5405443E-03 .5354455E-03 .5250556E-03 .5332989E-03 .5702750E-03 .6073587E-03 .6238510E-03 .6206080E-03 .6103508E-03 .6169691E-03 .6381478E-03 .6520717E-03 .6714566E-03 .6881739E-03 .7035144E-03 .9026073E-03 .1395040E-02 .1866983E-02 .2094477E-02 .2098097E-02 .2192174E-02 .2231608E-02 .2086257E-02 .2015806E-02 .1548635E-02 .9869402E-03 .6678391E-03 .6082774E-03 .6744762E-03 .7337738E-03 .7454260E-03 .7357667E-03 .7227923E-03 .7170014E-03 .7205124E-03 .7282776E-03 .7337615E-03 .7338096E-03 .7294443E-03 .7236954E-03 .7189890E-03 .7159917E-03 .7146940E-03 .7282089E-03 .7638884E-03 .7962717E-03 .8088320E-03 .8044051E-03 .7944445E-03 .7877263E-03 .7862033E-03 .7871369E-03 .7968493E-03 .1002018E-02 .1350300E-02 .1307025E-02 .1028130E-02 .7854354E-03 .7021718E-03 .7427508E-03 .8128639E-03 .8602646E-03 .8831948E-03 .8819390E-03	

APPENDIX I: PEER Ground Motion Data Base RSN454_MORGAN_GIL337.AT2	APPENDIX II: PEER Ground Motion Data Base RSN1633_MANJIL_ABBAR--L.AT2
.8550543E-03 .8227065E-03 .8238852E-03 .8469193E-03 .8665915E-03 .8893469E-03 .1298720E-02 .2316794E-02 .3262479E-02 .3682931E-02 .3634865E-02 .3215010E-02 .2363017E-02 .1569581E-02 .1282618E-02 .1153542E-02 .1043850E-02 .9405553E-03 .8539491E-03 .7947730E-03 .7645929E-03 .7416117E-03 .7054887E-03 .6720511E-03 .6678520E-03 .6992071E-03 .7325467E-03 .7445870E-03 .7361660E-03 .7203231E-03 .7081332E-03 .7026987E-03 .7009483E-03 .6985629E-03 .6838173E-03 .4719530E-03 .1413370E-03 .4926042E-04 .5084985E-03 .1253024E-02 -.1846320E-02 -.2074984E-02 -.2048704E-02 -.1963860E-02 -.1933293E-02 -.1988180E-02 -.2056847E-02 -.2071963E-02 -.2042912E-02 -.2019407E-02 -.2033501E-02 -.2051151E-02 -.2052298E-02 -.2060685E-02 -.2073578E-02 -.2081787E-02 -.2082644E-02 -.2078931E-02 -.2074502E-02 -.2071392E-02 -.2069489E-02 -.2067696E-02 -.2065133E-02 -.2061649E-02 -.2057642E-02 -.2053520E-02 -.2049498E-02 -.2036250E-02 -.1825077E-02 -.1332399E-02 -.1061625E-02 -.1136614E-02 -.1282286E-02 -.1395453E-02 -.1184308E-02 -.6577449E-03 -.1370449E-03 -.1302741E-03 -.4364186E-03 -.6833775E-03 -.5716425E-03 -.4189522E-03 -.8562765E-03 -.1515398E-02 -.1942913E-02 -.2030129E-02 -.1913433E-02 -.1602575E-02 -.1259559E-02 -.1301931E-02 -.1571995E-02 -.1604109E-02 -.1172829E-02 -.6526800E-03 -.3546270E-03 -.1308068E-03 .2596014E-03 .6298760E-03 .8215730E-03 .8594629E-03 .8483267E-03 .8476190E-03 .8804877E-03 .9393824E-03 .9849471E-03 .9970912E-03 .9838857E-03 .9649919E-03 .9664711E-03 .9974846E-03 .1029792E-02 .1034515E-02 .8388763E-03 .2150766E-03 -.3056247E-03 -.3718814E-03 -.5781998E-03 -.7253613E-03 -.6956394E-03 -.8133285E-03 -.1016610E-02 -.9737285E-03 -.3592771E-03 .5597845E-03 .1196423E-02 .1204110E-02 .8239262E-03 .7067939E-03 .7178446E-03 .3584341E-03 -.3549538E-03 -.9498595E-03 -.1171944E-02 -.1096513E-02 -.9468293E-03 -.8761509E-03 -.9154470E-03 -.9996267E-03 -.1039177E-02 -.1020300E-02 -.9637928E-03 -.9081965E-03 -.8764609E-03 -.6975478E-03 -.1190505E-03 .6924341E-03 .1266386E-02 .1433491E-02 .1341065E-02 .1212423E-02 .1171767E-02 .1215771E-02 .1268183E-02 .1269185E-02 .1228986E-02 .1182159E-02 .1152310E-02 .1142648E-02 .1143266E-02 .1142990E-02 .1136628E-02 .1125866E-02 .1125312E-02 .1132675E-02 .1121564E-02 .1098057E-02 .1076492E-02 .1063263E-02 .1056429E-02 .1051036E-02 .1043677E-02 .1033900E-02 .1023073E-02 .1012664E-02 .1003218E-02 .9944288E-03 .9856913E-03 .9766166E-03 .9671734E-03 .9575641E-03 .9479779E-03 .9385104E-03 .9148866E-03 .5871759E-03 -.1952157E-03 -.7483199E-03 -.5097890E-03 .2663395E-03 .9169568E-03 .1137731E-02 .1031408E-02 .8331977E-03 .7250517E-03 .7426655E-03 .8309866E-03 .9152619E-03 .9434528E-03 .9189103E-03 .8763286E-03 .8457889E-03 .8357699E-03 .8202896E-03 .6415972E-03 .4481974E-03 .3305219E-03 .3013324E-03 .1336267E-03 .1110653E-04 .2644792E-03 .4484343E-03 .2901724E-03 -.1734629E-03 -.7258945E-03 -.1059171E-02 -.1111185E-02 -.1013999E-02 -.9102752E-03 -.8806974E-03 -.9243532E-03 -.9816398E-03 -.9904933E-03 -.7918292E-03 -.3898150E-03 .8823135E-04 .3740107E-03 .4333559E-03 .3303875E-03 .1042452E-03 .8951491E-04 .3650959E-03 .6678296E-03 .8107759E-03 .7910941E-03 .7055964E-03 .6427475E-03 .6317619E-03 .6527730E-03 .6729201E-03 .6737466E-03 .6568595E-03 .6341752E-03 .6159374E-03 .6050643E-03 .5985868E-03 .5921196E-03 .5832343E-03 .5720859E-03 .5601888E-03 .5488434E-03 .5384313E-03 .5285795E-03 .5187424E-03 .5086068E-03 .4981954E-03 .4894476E-03 .4929000E-03 .4924152E-03 .4884416E-03 .4739651E-03 .4505319E-03 .4409597E-03 .4340158E-03 .4199745E-03 .4145616E-03 .3862879E-03 .1648749E-03 -.3347646E-03 -.8482025E-03 -.1112694E-02 -.1124196E-02 -.1028077E-02 -.9565950E-03 -.9536779E-03 -.9916245E-03 -.1024724E-02 -.1028614E-02 -.1007340E-02 -.9787078E-03 -.9571917E-03 -.9244945E-03 -.7253063E-03 -.4468885E-03 -.4203316E-03 -.6474214E-03 -.8859167E-03 -.9648178E-03 -.7459378E-03 -.3657202E-03 .5209818E-04 .3555206E-03 .3210986E-03 .1761046E-03 .3507146E-04 .5083188E-04 .2117496E-03 .2168378E-03 .3057301E-04 -.1861616E-03 -.4464227E-03 -.6891322E-03 -.8105666E-03 -.7974573E-03 -.5722363E-03 -.2039334E-03 -.1227625E-03 -.1844637E-03 -.6316049E-04 .1862182E-03 .3782237E-03 .4302562E-03 .3460392E-03 .7024057E-04 .3560079E-03 .6969990E-03 .8268563E-03 -.7988712E-03 -.7279361E-03 -.6916849E-03 -.7011646E-03 -.7274619E-03	

APPENDIX I: PEER Ground Motion Data Base RSN454_MORGAN_GIL337.AT2	APPENDIX II: PEER Ground Motion Data Base RSN1633_MANJIL_ABBAR--L.AT2
-7407802E-03 -7313504E-03 -7074911E-03 -6826692E-03 -6649035E-03 -6542288E-03 -6462618E-03 -6369204E-03 -6248367E-03 -6109742E-03 -5969900E-03 -5836669E-03 -5380836E-03 -3798523E-03 -1950155E-03 -1202456E-03 -2396303E-03 -4325420E-03 -5495481E-03 -5279699E-03 -3482697E-03 -1687897E-03 -1828581E-03 -3036978E-03 -3332229E-03 -3111783E-03 -1902936E-03 -2580292E-04 -1331675E-03 -1130547E-03 -1046171E-03 -7301106E-04 -1161732E-04 -1685196E-03 -3195950E-03 -3904046E-03 -3856043E-03 -3510960E-03 -3251627E-03 -3187310E-03 -3226132E-03 -3213989E-03 -3083992E-03 -2939477E-03 -2826690E-03 -2714903E-03 -2576997E-03 -2169433E-03 -1375147E-03 -6060971E-04 -9138826E-04 -2459207E-03 -3241351E-03 -3257188E-03 -2943273E-03 -2689480E-03 -2624652E-03 -2669684E-03 -2440079E-03 -1324318E-03 -7600555E-05 -1479140E-04 -4954233E-04 -1107223E-03 -1582086E-03 -1740891E-03 -1443419E-03 -3846791E-04 -1094548E-03 -2148471E-03 -2467573E-03 -2114416E-03 -1435151E-03 -1279768E-03 -1607230E-03 -1970713E-03 -1935732E-03 -1462628E-03 -1090335E-03 -5469363E-04 -1315063E-04 -4471023E-04 -5223070E-04 -2270924E-04 -1675281E-04 -3806249E-04 -3801821E-04 -4112271E-04 -7541581E-04 -1186342E-03 -1430577E-03 -1313772E-03 -6874975E-04 -1875513E-04 -6815909E-04 -4857862E-04 -1037423E-04 -2412850E-06 -2841761E-05 -7644428E-05 -1210940E-04 -1492171E-04 -1637988E-04 -2524756E-04 -5160156E-04 -6923747E-04 -4604657E-04 -5300360E-06 -2849345E-04 -4211606E-04 -4645167E-04 -4495120E-04 -4184216E-04 -3959818E-04 -3875078E-04	